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ERRATA.

Transactions, American Society of Civil Engineers, Vol. XXIII, July, 1890.

Page 2, 16th line from top, for "$= \frac{Px}{I}$" read "$a = \frac{Px}{I}$."

Page 10, after last formula, add "$p_o = \frac{P}{S} = \frac{\text{total stress}}{\text{area}} = \frac{P}{2x}$ for unity in width."

Page 12, 3d line from bottom, for "$\frac{2x}{3}\left(2 - \frac{pn}{P}\right)$" read "$\frac{2x}{3}\left(2 - \frac{px}{P}\right)$."

Page 13, 3d line from top, for "$q = \frac{2P}{3P}$" read "$n = \frac{2P}{3P}$;" and last formulas should read

$$q = \frac{x - n}{2x} = \frac{1}{2} - \frac{n}{2x}$$

$$n = x - 2qx = x\,(1 - 2q).$$

Transactions, American Society of Civil Engineers, Vol. XXIII, August, 1890.

Page 105, 2d line from top, for "intermediate" read "relative."

Page 108, 4th line from bottom, for "interposition" read "relative position."

Transactions, October, 1890, page 208, 5th line from top, for "generators" read "motors."

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442.

Vol. XXIII.—July, 1890.

THE STABILITY OF LOADED MASONRY ARCHES.

By ARTHUR S. C. WURTELE, M. Am. Soc. C. E.

During the examination of the Assembly Chamber groins in the New State Capitol at Albany, N. Y., it having become my duty to investigate the stability of the same under loads shown by exact measurements to be imposed thereon, I made a study of the conditions of stability of loaded arches, and state the results as briefly as possible, making no claim to originality, but giving the mathematical deductions of Rankine and others.

Given the form of intrados and loading as to weight and position, how is the stability to be shown?

In the first place, as the middle third of the arch-ring holds an important place in the stability of arches, I proceed to show the reason for

taking the middle third only into consideration with reference to line of pressure.

Let p = uniform intensity.
P = total load.
S = rib area.
a = uniform increase of varying intensity.
I = moment of inertia of section.
x = any distance from center of figure of rib.
x_1 = distance of extreme intensity.
p_1 = maximum intensity.
b = width of rib.
d = depth of rib.

The question is, at what distance will pressure change to tension?

$$p = \frac{P}{S} = \frac{P}{bd} \ldots\ldots\ldots (1)$$

$Px = aI$ and therefore

$$= \frac{Px}{I} \qquad ax_1 = \frac{Pxx_1}{I}$$

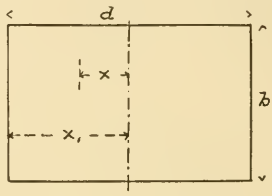


Fig. 1.

The maximum intensity $p_1 = p + ax_1 = p + \frac{Pxx_1}{I}$, and where a change occurs to tension, $p_1 = 0$. Therefore

$$p - \frac{Pxx_1}{I} = 0 \quad \text{or } p = \frac{Pxx_1}{I} \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$I = \frac{bd^3}{12}$ $x_1 = \frac{d}{2}$. Substituting these values and that of p from (1), we obtain

$$\frac{P}{bd} = \frac{Pxd}{2} \times \frac{12}{bd^3}, \text{ whence } x = \frac{d}{6}$$

or, the point where pressure would change to tension is one-sixth of the depth of the rib measured from the center of figure; and if the line of pressure is taken within that limit or within the middle third, there will be no tensile strain developed within the arch-rib; a very important point, as tension cannot be considered in a stone arch with any certainty or safety.

I now proceed to show how a line of pressure can be practically drawn from given loads, according to the system of Dr. H. Scheffler.

Let Fig. 2 represent the arch and loading reduced to the same specific gravity as the ring-stones. Form a table of areas or loads; distances of centers of gravity of loads; products, and sums; the results being moments of total loads to each joint and center of gravity distances of

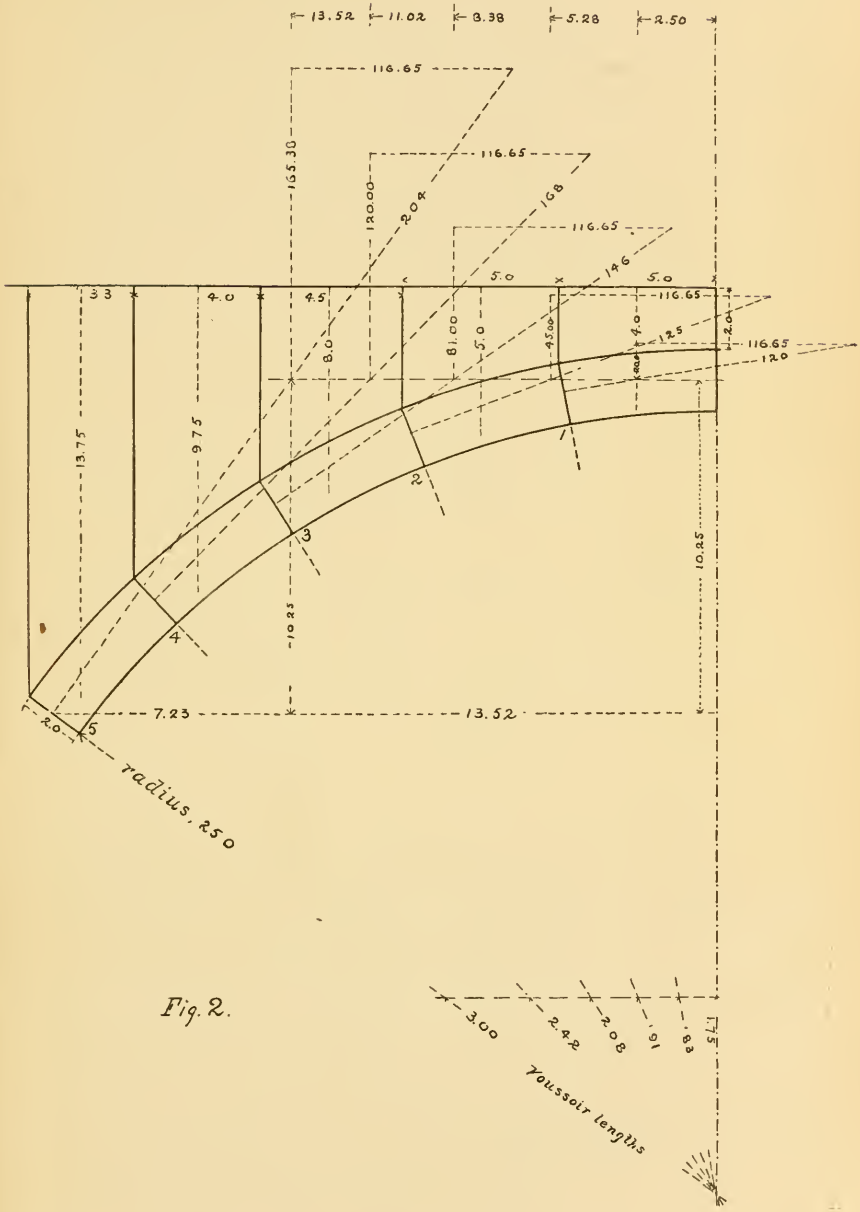

13.52
11.02
8.38
5.28
2.50
116.65
165.30
120.00
204
168
146
5.0
3.3
4.0
4.5
8.0
81.00
45.00
4.0
2.0
125
120
13.75
9.75
10.25
7.23
13.52
2.0
radius, 250
3.00
2.42
2.08
1.91
1.83
1.75
Voussoir lengths

Fig. 2.

total loads from crown. The final distance subtracted from the span and the result multiplied by total load and divided by the rise will give the horizontal thrust for that line of pressure, which graphically laid down horizontally at distance above the center of crown equal to total load on center of gravity vertical of such load will form a triangle, the hypothenuse giving the pressure on joint and, produced to said joint, giving a point in the line of pressure. This process repeated for each total load will give the completed line of pressure by joining points of intersection, thus forming a polygon which will approximate to a curve according to the number of joints taken.

Joint.	b	d	A	g	gA	$\int A$	$\int gA$	$\frac{\int gA}{\int A}$
1	5.0	4.00	20.00	2.50	50.00	20.00	50.00	2.50
2	5.0	5.00	25.00	7.50	187.50	45.00	237.50	5.28
3	4.5	8.00	36.00	12.25	441.00	81.00	678.50	8.38
4	4.0	9.75	39.00	16.50	643.50	120.00	1322.00	11.02
5	3.3	13.75	45.38	20.15	914.41	165.38	2236.41	13.52

$H = \frac{7.23 \times 165.38}{10.25} = 116.65$ gives the horizontal pressure $= \frac{d \times A}{h}$

The depth of the keystone can be obtained by Rankine's approximation of $d = \sqrt{.12\,R}$ (in underground arches co-efficient = .27 to .48 instead of .12 according to material), where R is the radius at crown or by Trautwine's formula, $d = \frac{\sqrt{R + \frac{S}{2}}}{4} + 0.2$ where S is the span. The depth of voussoirs is then proportional to the secants of the angle with the crown of the arch, and can be practically obtained by drawing a line across radii ruled to joints parallel to tangent at crown, at a distance from center of radii equal to depth of keystone.

In the case of arches unequally loaded as to halves, there will be a vertical component at crown, given as follows:

$V = \frac{d\,h'\,A - d'\,h\,A'}{yh' + y'\,h}$, y being the half-span and primes denoting opposite arch figures.

Then the horizontal pressure will be

$$H = \frac{d \times A - y\,V}{h}$$

Having examined the question of the middle third of the voussoir and the lines of pressure by a practical operation, we now pass to the main question of the stability of a given arch.

In order that a condition of stability may exist in an arch, it must be possible to construct a line of pressure falling everywhere within the middle third of depth of the voussoirs, as otherwise the condition of tension would exist, which cannot be allowed, as already stated.

It may also be held in most practical examples that the tenacity of the mortar is not sufficiently great or certain to be taken into account, and hence all horizontal or conjugate forces which maintain the equilibrium of the arch-ring must be pressures acting upon the arch-ring from without inwards, and consequently, the intensity of the horizontal or conjugate pressures must not at any point be negative.

It is always possible to determine a system of horizontal or conjugate pressures, which being applied to an arch loaded in any way will keep it *in equilibrio.*

Rankine gives a graphical solution as follows:

Given arch CC_5 loaded with $COO_5\ C_5$.

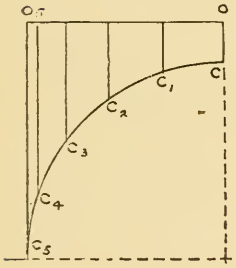
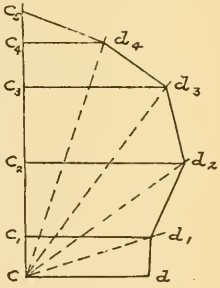


Fig. 3.



Draw line of loads, CC_5 vertical and lay off CC_1, CC_2 .... equal to total load on CC_1, CC_2 .... Fig. 3. At C draw lines Cd_1, Cd_2 .... parallel to tangents to arch-curve at C, C_1, C_2 .... and draw Cd, C_1d_1, C_2d_2 .... horizontal from C, C_1, C_2 .... on load line, to intersection with lines Cd_1, Cd_2 .... Then Cd, C_1d_1, C_2d_2 are the horizontal pressures and Cd, Cd_1 .... are the normal pressures on the joints C, C_1, C_2 .... measured on the same scale as used for the load line.

The pressure at the crown cannot be obtained graphically, but must be derived from the formula $p_1 = p \times r$, where p is the intensity of load at the crown and r is the radius at the crown.

The point where Cd or the horizontal pressure is greatest is called the joint of rupture and the horizontal pressure at any point C is the total thrust to be carried below that point to the springing-line.

The horizontal resistance required in any layer may be obtained by working out as above the total pressures at top and bottom of such layer, and the difference will be the resistance required.

The above results are expressed analytically thus:

Horizontal pressure $= H = P \frac{dy}{dx} = P$ cotan i, where i is the angle of the tangent at $C\,C_1$ ....

Joint pressure $= T = \sqrt{P^2 + H^2} = P$ cosec i.

Pressure on layer $= H - H_1 = - dH$, of which the intensity is $-\frac{dH}{dx} = -\frac{d}{dx}\left(P \frac{dy}{dx}\right)$, and which intensity must never be negative for stability, as before stated.

Another equation of condition for stability is :

$WdT > TdW$, where W is the load between crown and joint and T is the tangent of the angle of the joint. This must hold good at every joint, or there must result tension in the arch-ring, which cannot be allowed.

The intrados and extrados of an arch being given, it is possible to arrange the parts so as to be consistent with the laws of equilibrium on the supposition that there is no cohesion and no friction.

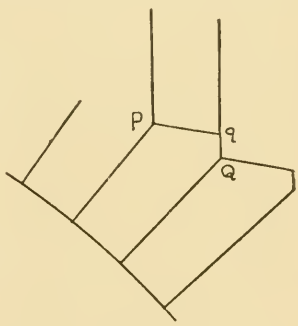


Fig. 4.

Let $P\ Q$ be the inner end of an arch-stone; the part $q\ Q$ being vertical and $P\,q$ inclined at an angle with the horizontal, whose tangent is Θ, and let the tangent of the joint angle P with the vertical be t.

The horizontal strain at P is $\frac{w}{t}$ and at Θ it is $\frac{w + dw}{t + dt}$ where if $P\ q$ is horizontal these would be alike, but the obliquity of $P\ q$ causes the load dw on it to generate a horizontal pressure Θdw, or

$$\frac{w}{t} - \frac{dw + w}{dt + t} = \Theta dw. \quad \text{Therefore,}$$

$$\Theta = \frac{w}{t^2} \quad \frac{dt}{dw} - \frac{1}{t} \quad \text{and} \quad \Theta t = \frac{w}{dw} \quad \frac{dt}{t} - 1$$

Now, intrados and extrados being given, w, t, dw, and dt are deducible and Θ may be computed by the help of differentials only.

This solution of the condition of equilibrium gives data whereby the inner ends of arch stones may be shaped, and even if the stones are not cut as required, it affords a means of deciding upon the practicability of any design, for if at any place Θt comes out with the wrong sign, that is, if $w.dt$ be less than $t.dw$, the building is unsafe and insecure.

It is allowable to take the springing-line at the joint of rupture, but in an arch so calculated it is absolutely necessary that the abutment and filling-in of spandrel be sufficiently stable to resist the horizontal pressure in every point below such joint of rupture. This condition is fulfilled in an abutment solid up to that point, as such an abutment will react with only just the pressure required; any other condition of pressure from the outside requires that such pressure should be exactly equal to the pressure of the arch, otherwise motion will finally result.

For an arch considered as above, the horizontal thrust is the horizontal thrust of the whole arch, and between the joint of rupture and the crown the condition of the line of pressure is that of vertical forces with horizontal component constant and equal to that at the joint of rupture, and the only important point to determine in the line of pressure is that at the crown of the arch, which may be obtained as follows:

Draw a vertical line through the center of gravity of the load between the joint of rupture and the crown, and if from any point in that line lines can be drawn parallel, respectively, to tangents at the joint of rupture and at the soffit at crown, both falling inside the middle third of ring, at joints, stability will be assured; always provided that the abutments are solid, and in no other case.

In pointed and elliptical arches it is sometimes impossible to obtain a line of pressure within the middle third, when the correct process is to obtain by trial the longest line of pressure possible within the middle third of the ring, which will give the minimum horizontal pressure which can be used, and give the greatest stability possible. The point where said longest line of pressure runs out of the middle third will be the joint of rupture.

The arch-ring below the joint of rupture must be sufficient to carry the horizontal strain, and is to be made so by increasing the depth of the arch-ring, and below above taken springing-line the abutment must be sufficient to resist the horizontal pressure.

If the above conditions are not fulfilled, the joint of rupture will open at back end; the joint at the springing-line will open at the front. If we provide an abutment capable of reacting with a force sufficient to keep the equilibrated polygon, where it cuts the joint, within the middle third of the arch-ring, our abutment will certainly be strong enough, and that horizontal force is to be adopted in all practical calculations.

There appears to be no condition under which the springing-line can be taken at any other point than the actual springing-line; except with the absolute condition of a solid abutment, and even then the horizontal pressure must be calculated from the whole arch.

It can be inferred from the differential equations of condition that regularly increasing the loading from the crown down is an important condition of stability, as a sudden increase of load at one point is liable to give a negative equation, which implies tension in the arch-ring and consequent danger of final disintegration.

According to Rankine, in a complete rib, rising vertically from the springing-line, the whole horizontal pressure to sustain it will be distributed through the layers of the spandrel; but in a segmental rib, rising obliquely from the springing-line, only the excess of the greatest horizontal thrust over the horizontal component of the thrust along the rib will be so distributed. Thus, if P is the vertical load between the crown and the springing-line; i the inclination of the rib at the springing-line; H the greatest horizontal thrust: then $H = P \cot i$ will be the thrust distributed through the spandrels.

Sometimes the curve of a line of pressure from given loads may be adopted as the intrados of an arch, and the practicability of this may be tested as follows:

If $A\ C\ B$ be the line of pressure given, take Aa, normal at the crown, and not over two-thirds of the depth proposed for the keystone; take Bb, also normal at the springing-line, and of such length that

$$\frac{Bb}{Aa} = \frac{\text{Thrust along rib at } A}{\text{Thrust along rib at } B}$$

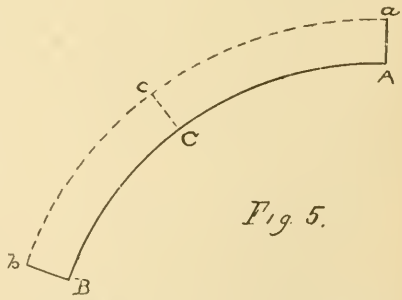


Fig. 5.

Draw in the line $b\ c\ a$ at such a distance from $B\ C\ A$ that the

normal at any point, as $C\,c$, shall be inversely as the thrust at that point. Then by conceiving couples of equal moment of tension at A and a thrust at a equal to the thrust along the rib at A and a tension at B with a thrust at b equal to that at B; these being equal and opposite will only transfer the line of pressure from $A\ C\ B$ to $a\ c\ b$, so that if $a\ c\ b$ is within the middle third, then $A\ C\ B$ is suitable for the intrados of the arch.

I conclude with a simple graphic determination of an equilibrated circular arch, to show relative depth of voussoirs.

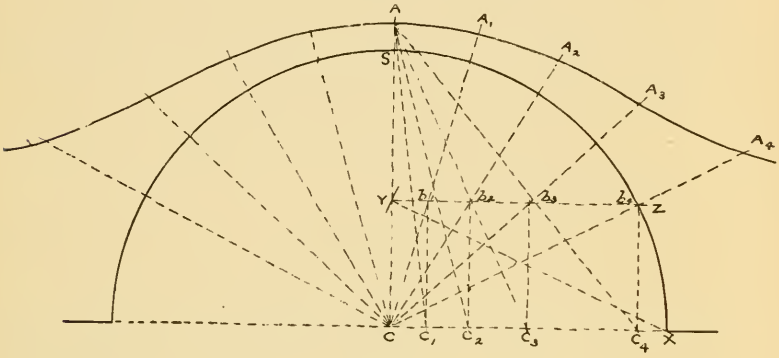


Fig. 6.

Let $S\,A$ be the depth of the keystone. Then from X as a center, with a radius $C\,A$, cut $C\,S$ in Y and draw $Y\,Z$ horizontal; draw radius $C\,A$; make $b_1\ c_1 \ldots b_2\ c_2 \ldots . bn\ c_n$ parallel to $C\ Y$, and finally take

$$C\,A_1 = C_1\,A, \ldots\ldots C\,A_n = C_n\,A.$$

Join points $A\ A_1\ A_2 \ldots\ldots$ and the line $A\ A_1\ A_2 \ldots\ldots$ will represent the extrados of an equilibrated circular arch.

This investigation of the equilibrated arch is written after examination of Rankine's works, the Encyclopædia Britannica and other works, combined with some practical experience in construction.

SUPPLEMENT.

Uniformly Varying Stress in Foundations for Reservoir Walls and Abutments to Arches.

This stress is such that the intensity at a given point is proportional to the distance of that point from a given line. Thus, let $p = ax$ represent the law of variation of intensity, a being constant.

The amount of stress is given by the equation

$$P = \int\int p\, dx\, dy = a \int\int x\, dx\, dy$$

The center of stress is the point traversed by the resultant of the whole stress, and may be obtained by dividing the moments of differentials of stress by the sum of such differentials.

$$x_0 = \frac{\int\int x\ p\ dx\ dy}{\int\int p\ dx\ dy} = \frac{\int\int x\ p\ dx\ dy}{P}$$

but $p = ax$, and therefore

$$x_0 = \frac{a \int\int x^2\, dx\, dy}{P}$$

and $\int\int x^2\ dx\ dy$ is known as I or the moment of inertia, so that finally, $x_0 = \frac{a\,I}{P}$ = distance of the center of pressure of uniformly varying stress from the center of figure.

Uniformly varying stress may be considered as made up of two parts:

First.—A uniform stress whose intensity is the mean intensity of the stress and whose center is the center of gravity of the surface under stress.

Second.—A uniformly varying stress whose neutral axis traverses the above center of uniform stress and whose amount is zero and intensity p at a given point is the deviation from the mean at that point, so that the intensity of the entire stress is

$$p = p_0 + p_1 = p_0 + ax_1 \text{ and}$$

$$a = \frac{p - p_0}{x_1}$$

We know by the theory of couples that a force P applied at a distance x_0 from the center may be replaced by an equal force at the center and a couple of moment Px_0, the effect of the couple being to shift the

resultant through a distance equal to the arm of the couple, so that $Px_o = aI$ as before shown for uniformly varying stress.

The non-axial force may be assumed to produce a uniformly varying stress, the maximum intensity of which will occur at the distance x_1 from the neutral axis on the same side as x_o is taken. This uniformly varying stress is equivalent to a uniformly distributed stress of the intensity p_o and a couple of the moment aI where a is the constant rate of increase of the stress.

Let q = relative deviation of center of pressure from the center of figure to total depth ; x_1 half-depth ; and I for rectangles $= \frac{2x_1^3}{3}$; when the width is unity ; $q = \frac{x_o}{2x_1}$; and by combining the above equations, we obtain

$$q = \frac{aI}{2Px_1} = \frac{ax_1^2}{3P} = \frac{(p - p_o)\,x_1}{3P} = \frac{2px_1 - P}{6P} = \frac{px_1}{3P} - \frac{1}{6}$$

from which

$$p = \frac{P\,(6q + 1)}{2x_1}$$

These equations form a general solution, but are limited by the condition that the stress is of the same kind over the whole surface. This limit is found as follows :

In the equation $p = p_o + ax_1$ the minimum intensity is $p = p_o - ax_1$; therefore, when ax_1 is greater than p_o, it becomes a minus quantity, showing tension on a part of the surface, and $p_o = ax_1$ gives the limit, or

$$a = \frac{p_o}{x_1}$$

which combined with the equation

$$q = \frac{aI}{2\,Px_1} = \frac{a\,x_1^2}{3\,P} \text{ gives}$$

$$q = \frac{p_1\,x_1}{3P} = \frac{P\,x_1}{2\,x_1.\,3P} = \frac{1}{6}$$

from which it is determined that when a uniformly varying stress increases from zero up, q must be one-sixth of the depth, and from equation:

$$p = \frac{P\,(6q + 1)}{2x_1} \text{ we obtain the value}$$

$$p = \frac{2P}{2x} \quad \text{or,} \quad p = 2\,p_o.$$

So that, when q exceeds one-sixth, the surface of pressure will be shortened and q always one-sixth of the shortened length, and p always double p_o.

General equations for this condition may be derived as follows:

Let z = depth of surface pressed,
x_1 = half total depth.

Then $2\,x_1 - z$ = depth under tension, half of which, $x_1 - \frac{z}{2}$ = distance between the centers of total and pressed depth. Therefore,

$$2\,x_1\,q = x_1 - \frac{z}{2} + \frac{z}{6} = x_1 - \frac{z}{3} \text{ whence } z = 3\,(x_1 - 2\,x_1\,q)$$

$$p_o = \frac{P}{2} \text{ and } p = \frac{2\,P}{z}. \text{ Therefore,}$$

$$p = \frac{2\,P}{3\,(x_1 - 2\,x_1\,q)} = \frac{2\,P}{(3 - 6\,q)\,x_1} \text{ and}$$

$$q = \frac{1}{2} - \frac{P}{3\,p\,x_1}.$$

These equations of condition are of great use in matters of foundations, reservoir walls and all structures with deviations in center of pressure; so that a summary of above results will be advantageous for reference.

Greatest Intensity and Deviation from Center of Pressure in Foundations.

First.—When the deviation is less than one-sixth depth of base, or nearest edge more than one-third depth from center of pressure.

Let P = total pressure on unit width of base;
p = greatest intensity of pressure;
x = half-depth of base;
q = deviation of center of pressure from center of base to base;
n = distance of center of pressure from nearest edge of base.

Then,

$$p = \frac{P\,(6\,q + 1)}{2\,x}; \text{ or, } p = \frac{P}{x}\left(2 - \frac{3\,n}{2\,x}\right)$$

$$q = \frac{p\,x}{3\,p} - \frac{1}{6} \qquad n = \frac{2\,x}{3}\left(2 - \frac{p\,n}{P}\right)$$

Second.—When the deviation is greater that one-sixth depth of base, or nearest edge less than one-third depth from center of pressure.

Employing the same notation as above, we obtain

$$p = \frac{2P}{(3 - 6\,q)\,x}; \qquad \text{or,} \qquad p = \frac{2P}{3n}$$

$$q = \frac{1}{2} - \frac{P}{3p\,x} \qquad\qquad q = \frac{2P}{3p}.$$

The form in terms of q is given by Rankine, and that in terms of n is given by J. B. Krantz, and conversion is easy by the following equations:

$$q = \frac{x - n}{2\,x} = \frac{1}{2} = \frac{n}{2\,x}$$

$$n = x - 2 = x\,(1 - 2\,q).$$

DISCUSSION.

William Cain, M. Am. Soc. C. E. — This investigation of the masonry arch goes no further than Rankine, though considerable advance has been made since his time in the theory of the arch. See, on this subject, an article by Professor George F. Swain, in *Van Nostrand's Magazine* for October, 1880, where various authors are quoted as referring the theory of the stone arch, under certain limitations, to that of the solid elastic arch "fixed at the ends."

Some years ago the writer, when a young assistant engineer, had occasion, as Mr. Würtele has had, to study the arch question, but, not being satisfied with Rankine, took up Scheffler's theory of fictitious incompressible voussoir arches, and modified it empirically to make a practical solution of the arch shown on page 104 of Van Nostrand's "Science Series," No. 12 (published in 1874), limiting the curve of pressures to the middle third of the arch-ring to avoid any tension, as Rankine suggests. It is very evident from the constructions given that the consideration of a uniform load alone (as with Rankine) is worth little as a test of the strength and stability of an arch, but that for rolling loads, the arch partially loaded is much the surer and only true test. This is so plain from the constructions that it is surprising that any other views should be entertained. As an approximate solution and fairly good test of the stability of any arch subjected to a rolling load, I should use now very much the construction alluded to for any segmental arch, or for the portion of a full center arch above where the spandrels are supposed to be built so solidly as to be as immovable as the abutment, and, in fact, to constitute a part of it. If the spandrels give appreciably, the solution is indeterminate by any theory whatever, unless the exact resistances

furnished by them could be known beforehand. The proper way to avoid this ambiguity is to increase the depth of voussoir as we leave the crown and continue the arch into the abutment of such dimensions that there is no need for a spandrel resistance to keep the centers of resistance within the middle third, or even narrower limits.

Further reflection led the writer to refer the theory of the voussoir arch to that of the "solid arch fixed at the ends," as announced in 1879, in "Science Series," No. 42 (page 41), and No. 48 (page 65), provided the centers of pressure all lie within the middle third and there are no mortar joints. In fact, conceive a solid arch of stone between immovable abutments of such dimensions that the true pressure curve lies everywhere within the middle third, so that no tension is exerted anywhere, the spandrel resistance, if any, being neglected. Now if this arch is conceived to be cut into any number of voussoirs, the deformation, which is due to the compression throughout, is precisely the same as before, hence the line of pressures for such a voussoir arch is precisely the same as for the corresponding solid arch, as it is determined entirely by this deformation, so that the graphical construction for fixing its position is the same in either case. Where very thin mortar joints are interposed which are allowed to harden before the center is struck, the conditions are not materially altered, and the theory of solid arches again applies very nearly if we consider the whole arch to be made of stone. Where the above conditions do not hold, then the problem becomes more or less indeterminate; but in a well-designed arch very thin mortar joints, or none at all, should be the rule, and the arch should be deep enough so that the true line of pressures should always keep within the middle third for any loading, in which case the theory of solid arches applies, and the spandrel resistance, if any, should be regarded as only adding to the stability of an otherwise safe structure. The same theory of solid arches likewise applies when temperature stresses are considered. As the proper constructions in these cases have been given by the writer elsewhere it is not necessary to enter into them here.

From a number of such constructions, particularly for uniform loads and to a somewhat less extent for partial loading and other considerations, the writer felt authorized, in 1879, to announce that "it seems highly probable that the actual line of pressures is confined within such limiting curves, approximately equidistant from the center line of the arch-ring; that only one curve of pressures can be drawn therein, corresponding, therefore, to the maximum and minimum of the thrust in the limits taken." Now if the limits are the middle third limits it is a very easy matter to test the stability of an arch for any loading after the method given in "Science Series," No. 12, page 104; and in fact, by a tentative method, to design an arch to satisfy the middle third or even narrower limits. I hope some time to design a series of arches of different spans for heavy railroad locomotives after

this simple test; and I feel sure that when it is done by myself or any one else, Rankine's and Trautwine's rules for depth of keystone will be found to give too small dimensions. In fact, it stands to reason that these rules, which simply represent the average of practice for the old highway bridges principally, should give too small dimensions for the enormous weights of our engines of to-day, traveling, too, at 40 miles per hour and upwards. Devotion to empiricism has been carried a little too far in this case.

In the article of Professor Swain, above alluded to, is given the demonstration of a remarkable theorem published by Winkler in 1879: "For an arch of constant section that line of resistance is approximately the true one which lies nearest to the axis (center line) of the arch-ring as determined by the method of least squares."

This theorem agrees in a general way with the principle mentioned above, and if any practical use is to be made of it, it would seem that we ought to work to "the principle," though it is admitted that the two are not identical and probably do depart from each other sensibly in the case of heavy eccentric loads, as I infer from some constructions for stone arches given by Castigliano in his "Systemés Elastiques" (1879-80), who adopts, without question, the theory of the "solid arch fixed at the ends" as the true one for voussoir arches, the curves of pressure, however, being located by the principle of "least work" in place of by usual methods.

It is not difficult, though it is tedious, to locate the true curve of pressures, after the theory of the solid arch, by the graphical method; but it is an open question whether "the game is worth the candle" when we consider how imperfectly the voussoirs are cut in practice. In the St. Louis steel arch, for example, the greatest care was observed to have the lengths of the main members minutely accurate, so that the theory would not give erroneous results. In the voussoir arch, as ordinarily constructed, the stones are by no means cut with minute accuracy, so that when the keystone is reached it is impossible to say whether it has just the size necessary for the closest fit, and yet on this depends the actual resulting line of pressures. In fact, pieces of slate are sometimes driven in between two voussoirs that do not fit, thus altering the resulting line of pressures. If, in addition, the mortar joints are not allowed to harden and the centers are struck as soon as the arch is turned, it would seem that the strict theory could only approximately apply and that the rough method given above would afford almost as good a test of the strength and stability of the arch as the more precise one, particularly as it is very near it in most cases.

Mr. Wurtele—Mr. Cain is entirely correct in stating that I go no further than Rankine in investigation of arches. I consider Rankine's mathematical results as definite, and though there may have been since considerable advance in the theory of the arch, yet there is no change

or advance in the general principles of stability; and in fact I have a work by Bossu, published in 1802, which really contains the complete principles of stability stated generally. The advance is largely based on hypothesis and results "highly probable" and often indeterminate; and, in fact, the only case really determinate is an arch built of blocks without any cohesion whatever. And my paper was only intended to give a practical way of determining the stability of an arch, without any consideration of Castigliano's "Systemés Elastiques," or Winkler's constant sections determined by least squares, which last I have a high respect for; but after all, least squares are not definite, as every new determination by least squares differs from a former determination by least squares.

The action of a rolling load can be determined by unequally loaded half-arches, as in a large arch the weight of the arch is many times the weight of rolling load, so that the effect is greatly diminished.

Mr. Cain's remarks are very interesting and valuable, as showing the possibilities and indicating the lines of reasoning when we come to a complete scientific reduction of the theory of the arch, and I am pleased that my little paper gives us a discussion so much more valuable than the paper itself.

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TUNNEL SURVEYING ON DIVISION No. 6, NEW CROTON AQUEDUCT.

By F. W. WATKINS, M. Am. Soc. C. E.

In surveying for shaft and tunnel work many difficulties are encountered that to the novice in underground work are somewhat embarrassing.

My first experience in underground work was on the Weehawken Tunnel of the New York, Ontario and Western Railroad. I searched through such books on tunneling as I could get access to, but did not find much light on the surveying question, those books being usually devoted to the more serious problems of tunnel engineering, timbering, etc.

In long lines of tunnel work, great care is necessary to transfer the center line, established on the surface, to the tunnel below, to insure the correct meeting of the headings.

Of course, all underground work must be done by the aid of artificial lights, and the atmosphere in a tunnel in course of construction is anything but favorable for accurate surveying.

In sinking shafts, even where a solid rock tunnel is expected, the upper strata of surface soil, sand, gravel, and disintegrated or shaly rock, usually contain more or less water, and extend a considerable depth before a solid, compact rock is met with, and must all be drained down the shaft to the sump at the bottom, so that when the engineer, with his instruments, arrives at the bottom of the shaft to give direction for the headings, he finds himself standing under a heavy rain, with his water-proof coat and rubber boots, but it is a serious question how he is to protect his instruments, so that the object glass, eye-piece and verniers can be kept free from moisture long enough for him to get a clear sight.

In tunnel headings, there frequently is considerable leakage, water is dripping from the roof, and the air is charged with vapor.

The smoke from numerous miners' lamps and torches, the gases and smoke from the frequent blasts of powder and dynamite used in driving the tunnel in rock, and the dry powder of the rock itself, thrown in the air by the power drills (when dry holes are being drilled), all combine to make the atmosphere very thick and hazy, and even with the best ventilating arrangements commonly used by contractors, these conditions are only partially remedied. I have seen it so thick at times that even an electric arc light, but 100 feet away, could not be distinguished but as a dim halo in the surrounding gloom.

Directly at the headings, where the working force is mostly employed, a better condition usually prevails. Immediately after a series of holes have been fired in the heading, by a liberal use of the compressed air, provided for running the power drills, the smoke and foul air are driven back a hundred feet or so from the heading, and then slowly make their way toward the shaft.

By continued use of powerful blowers or exhaust engines for several hours, the air in the tunnel can be rendered tolerably clear, so that sights of several hundred feet can be obtained, until another series of blasts are fired; the bad air must then again be blown out. When it is clear enough in the headings for his men to resume work there, the contractor is not much interested in driving it all out of the tunnel, and the continued running of the ventilating engine is so much more expense.

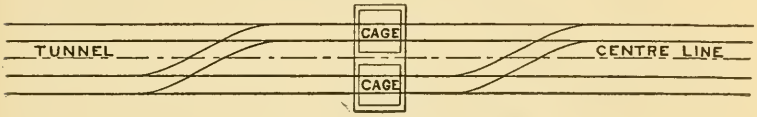
When construction work is carried on night and day, through the week, as is customary in long tunnels, any really accurate work of alignment can only be done there on Sundays or holidays, after the work on the headings has been stopped for some hours and the foul air blown out.

The Weehawken Tunnel of the New York, Ontario and Western Railroad was nearly 4 000 feet long, and was driven from five shafts, and the Eastern Portal, Shaft No. 5, when the approach cut was completed, becoming the Western Portal.

The following table gives the distance apart and depth of these shafts from the surface to the tunnel floor or grade line:

Shaft.	Station.	Distance Apart.	Depth.
E. Portal.....	2 + 70		
No. 1.....	10 + 44	744	146.7
No. 2.....	18 + 62.5	818.5	159.8
No. 3.....	26 + 80.5	818	161.2
No. 4.....	35 + 04	823.5	142.8
No. 5.....	42 + 55	751	90.7

The shafts were 8 x 16 feet, and placed with their longest side across the tunnel.



PLAN OF SHAFTS, WEEHAWKEN TUNNEL.

This arrangement gave the contractors access to both cages from their tracks on either side of the shaft, which was at times a convenience to them, but not actually necessary.

This arrangement of the shafts gave the engineers an effective base, for transferring the center line down the shafts, of about 7 feet 10 inches, allowing 1 inch on each side for clearance of timbers.

As the shafts averaged but about 800 feet apart, any error made in the alignment at the foot of the shaft would not be magnified over one hundred times at the next shaft, and it was expected would not amount to anything serious. As a matter of fact, the difference of lines when the various headings met ranged from ¾ of an inch to 1¼ inches. The apparatus used on this work was similar to that afterwards used on Division No. 6 of the New Croton Aqueduct Tunnel, but in a more crude form.

Division No. 6 of the New Aqueduct Tunnel (of which I had charge from the commencement of work on December 30th, 1884, to August, 1886) extended from a point near the north boundary of New York City to the siphon under the Harlem River, near High Bridge, and was 24 140 feet in length—all tunnel work.

Most of this tunnel was on the depressed portion of the Aqueduct which is to be under pressure, the grade line of the invert being about 130 feet below the flow line, or hydraulic grade of the Aqueduct.

This tunnel was planned to be driven from five shafts, No. 20 to No. 24, inclusive, to which another shaft was afterwards added.

The distance from Shaft No. 19 to No. 20 being very long, 6 650 feet, and much of it in bad ground, requiring timbering, Shaft No. 19½ was ordered and sunk at 3 116 feet north of Shaft No. 20.

The following table gives the distance between the shafts, and depth from the surface to the tunnel invert grade.

Shaft.	From Surface to Invert Grade—feet.	From Top of Curbing to Invert.	Distance between Shafts—feet.
No. 19½ ...	56.83	57.00	
" 20 ...	128.87	138.82	3 116
" 21 ...	115.21	135.51	5 426
" 22 ...	94.11	104.65	6 925
" 23 ...	78.21	86.17	3 940
" 24 ...	189.96	205.07	4 280

The working depth of most of the shafts was greater than the original depth, as the curbing had to be carried up, and waste rock from the shaft was filled in around it; at Shaft No. 20 as much as 20 feet, and at No. 24 for 15 feet higher than the original surface.

In the alignment there was one change of direction between Shafts Nos. 20 and 21 of 21 degrees 3 minutes. One change between Shafts Nos. 23 and 24 of 7 degrees 11 minutes, and just south of Shaft No. 24 of 72 degrees to the right, to go under Harlem River.

The grades changed from 0.7 feet per mile to 2 feet 9 inches per 100 feet, then to an incline of 10 feet per 100 feet for 1 000 feet, then to 0.7 feet per mile, then to 15 feet per 100 feet for 850 feet, and then to 2 feet per 100 feet, under the Harlem River.

The work of alignment on the surface for the New Aqueduct, from Croton Lake to the Harlem River, was all very carefully checked and monumented, under the direction of Mr. Alfred Craven, M. Am. Soc. C. E., and it remained for the division engineers to check this alignment,

locate the shafts, and carry the alignment from the surface, down the shafts, to the tunnel below.

Each division engineer was held responsible for the accuracy of the alignment, as well as for all other work on his division, and was left free to adopt the method he considered best, and though similar in the general plan, the details varied on each division.

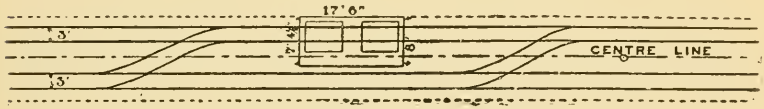
In such long headings, as between Shafts Nos. 20 and 21, nearly 7 000 feet, a variation or error of 1 inch in alignment at the foot of the shaft would be equal to 18 feet divergence where the headings were expected to meet.

In the Aqueduct Tunnel, where there were to be many long headings to be driven from shafts, the policy of placing the shafts with their greatest dimension in the axis of the tunnel was earnestly advocated by the engineer of construction, the late Mr. H. S. Craven, M. Am. Soc. C. E., and myself, and was so determined and fixed by the chief engineer before the contracts were let.

The dimensions of the large shafts were to be 17 feet 6 inches x 8 feet, and of the small shafts, 11 x 8 feet.

These shafts were placed longitudinally with the axis of the tunnel and on one side of the center line, being 7½ inches on one side and 7 feet 4½ inches on the other.

This was arranged so that the center-line wires would not interfere with the cages or elevators in the shaft, and also allow the contractors room for a second track or switch at the bottom of the shaft.



PLAN OF SHAFTS, NEW AQUEDUCT TUNNEL.

The space at each end of the shaft near the center line was reserved for the use of the engineers in plumbing down, and the contractors notified to keep it clear; but as there were two elevators to be put in each shaft, with bracing and guide timbers, steam pump and discharge pipes, compressed air pipes to convey power to each heading, two ventilator pipes, electric light and firing wires, a ladder down the shaft, etc., constant watchfulness was required of the assistant to see that this reserved space was not encroached upon by the superintendents, who

did not see any necessity for so much trouble about alignment, but if any error was made, of course the engineers were responsible.

The shaft being properly curbed at the top, and the work started, is easily kept vertical as the sinking progresses by plumb-lines suspended on the center line or at the four corners. Two pound lead plumb-bobs, suspended by braided fishing-line, were used for this purpose. When the grade of the tunnel is reached, the headings can be started and kept in line for 50 or 100 feet, by simply stretching a fine string parallel to the suspended center line plummet wires, and marking line points in the roof. (Plate I.)

When the construction work has progressed thus far, the engineer has then secured a point where he can place his instrument in the tunnel 30 or 40 feet from the shaft, clear of the water flooding down from above, and from a frequent danger of stones or blocks, etc., falling down the shaft.

His next step is to suspend his center-line wires one at each end of the shaft, secure them in the true line above ground, bring his plummets to a rest, just clear of the bottom of the shaft, and be absolutely certain that the wires do not touch anything from the top to bottom of the shaft, and then transit his instrument in line with the suspended wires, and mark his points in the roof of the tunnel from which to produce his line as the headings progress.

Above ground the transit was placed on line about 40 or 50 feet from the shaft, so as to look across the mouth of the opening to a line target, permanently secured, and as far away as could be distinctly seen.

The wire farthest from the instrument was first adjusted, and clamped in a little instrument with a tangent screw motion, and the nearest wire then adjusted and secured, care being taken not to change the focus or in any way shift the telescope until both wires were clamped with the same sighting.

The steel wire used was No. 8 piano wire, was annealed and wound upon a reel about 6 inches in diameter, and large enough to hold about 300 feet of wire, and provided with a clamp screw or brake.

The annealing of the wire was not perfect, and if at any time the tension upon it was released, it was apt to spring up the shaft, get tangled and break.

The reels were screwed fast on top of the head-frame of the shaft, a 1-pound plumb-bob attached and lowered to the bottom, where an assist-

ant took off the small plumb-bob and hooked on the 25-pound plummet weights, and by signals to the man at the reel on top, it was adjusted to be clear of the floor and hang in a bucket of linseed oil. The weight of these plummets was, therefore, carried to the top of the head-frame, but to avoid the vibration caused by alternately hoisting either cage the tangent screw instruments for securing the wires on the line were screwed to plank brackets projecting over the shaft and spiked to stout posts planted in the ground, free from the shaft timbering and the vibration caused by the hoisting machinery.

At the bottom of the shafts, the water continually dropping into the buckets kept the water or oil in them constantly agitated, preventing the plummets from coming to a rest. This was obviated by providing loose covers for the buckets, with a small space around the wire, enough to give it free play. This had to be closely watched, and every foot of the wires down the shaft closely scrutinized to see that it did not touch any of the timbers of the curbing and that there was no obstruction to prevent its hanging vertically. From the irregularities in setting the curbing many of these timbers had to be notched out to insure a clearance.

There was generally two or three days' preliminary work at each shaft getting the wires hung and cutting away to insure their hanging clear, and a trusty man was sent to climb from timber to timber to do this work before it was safe to go any farther.

The observer, on looking through his transit, sees three black lines, the vertical cross-hair of his telescope, and two plummet wires, and they appear so nearly alike that it is difficult to distinguish which wire is nearest him. Lights must be flashed behind each wire, and by the time he has moved his instrument nearly in line some one may have touched one of the wires and set it swinging, and he must wait for it to settle.

To obviate these and other difficulties, I devised what I have called an illuminated slit apparatus to replace the wires at the bottom of the tunnel, which proved to be very useful for this work. (Plate IV.)

This instrument consists of two vertical strips of brass about 3 inches high, attached to separate horizontal bars, moving in guides, and with a tangent screw motion, by which one or both could be moved right or left, and the vertical opening between them made as small as desired.

One of these instruments was screwed to a plank bracket, close

behind each plummet wire, and placed at such a height that on looking through the transit the farther one could be seen in line just above the other.

These vertical slits were then adjusted by the tangent screws to be directly behind the plummet wires, and with an opening slightly larger than the thickness of the wire, and so that a light held beyond it on line would illuminate an equal space on each side of the wire; this could be estimated very closely by the eye.

The plummets and wires were then removed, lights placed behind these slits, and two fixed and illuminated points substituted, which could be very closely bisected by the cross-hair of the transit, and transferred to the headings.

Line points in tunnel work are usually made in the roof, as any points made on the floor are liable to be moved or destroyed by the shifting of tracks, trampling of mules, cutting of drains, etc. In timbered ground a nail or screw-eye is put in one of the timbers, and a wire and plummet lamp hung from it; but such points are not very reliable, as the timbers may shift from unequal pressures. In rock tunnels a hole is drilled in the roof by a hand-drill, a wooden plug driven in and a screw-eye put in it; from that a copper wire and plummet lamp is suspended, and adjusted to the exact line. The copper firing-wires that have been used in the tunnel are excellent for this purpose; they are pliable enough to be easily straightened with a pull, and can be coiled up to the screw-eye in the roof and left to hang there out of the way until again wanted.

For test alignment work, a point was selected in each heading from 125 to 150 feet from the shaft, which would give an extended base line from 250 to 300 feet long.

Line points were then drilled in the roof 6 or 8 inches deep, wooden plugs driven in, and stout flat iron spikes, 9 inches long, driven into these plugs. To these spikes a horizontal brass plate was then bolted, with a zero mark cut on the lower edge; a movable vernier bar could then be hung under the brass plate, and the distance, east or west of the "0" mark, of a new test line could be read off. (Plate III.)

The whole process of carrying the alignment down the shafts and taking it up at the bottom was then gone through a number of times, and the variation of the vernier from the fixed points recorded; a mean of at least three good observations taken, the verniers set and clamped,

and wires hung from them; and from these points, 250 or 300 feet apart, the center line was extended into the headings, and all previous points given were corrected.

Record of Alignment Tests.

In Shaft No. 20 the test spikes were placed 150 feet north and 145 feet south of the center of the shaft, or 295 feet apart.

Three independent tests were made by plumbing down the shaft, and the reading of the vernier showing these lines to be east or west of the "0" on the test spikes recorded, as follows:

Variation of Test Line from Zero.

Date.			On North Spike.		On South Spike.		Observer.
September	27th,	1885.	1.00 inches	west.	0.30 inches	east.	F. W. W.
December	11th,	"	1.20	"	0.20	"	W. McC.
"	21st,	"	0.70	"	0.00	"	F. W. W.

The difference in direction or divergence from parallelism between the first and second observations was equal to one-tenth of one inch in 275 feet, equivalent to a variation at the base line of 17 feet, at the foot of the shaft of about $\frac{5}{10\,000}$ of a foot, and at 3 848 feet south of the shaft, or half-way to Shaft No. 21, the variation would be $1\frac{1}{6}$ inches.

The difference of direction between the first and third readings was $\frac{6}{10}$ of an inch in 295 feet. It was suspected that an error had been made at some stage of the observation, and it was thrown out, and the average of the first two taken. The verniers were then set for the north heading spike at 1.10 inches west of 0, and for the south heading spike at 0.25 inches east of 0 on the spikes, the wires and plummet lamps hung therefrom, and this base line extended into the headings, and all former points corrected.

Further tests could have been made, but were not considered necessary.

The north heading of Shaft No. 20 was carried 150 feet from the shaft, on a nearly level grade, and then changed to an incline, ascending 10 feet per 100, for 1 046 feet.

At the top of this first incline, and about 1 200 feet from the shaft, the surface of the ground was but 40 feet above the roof of the tunnel. A test hole, made by the diamond drill, when testing for rock, had been sunk at this point, and the casing pipe, 3 inches diameter, was left in

for the purpose of checking the alignment in the tunnel, and when the heading reached this point, though the pipe was not driven plumb, nor quite on line, it was calculated that our line below was 1¼ inches east of the line above ground.

At Shaft No. 21 the first series of observations were unsatisfactory, and considered to be approximate only. The contractors' men had placed a steam pump at the bottom of the shaft, on line, and the water and steam pipes crossed the line of sight, and interfered with the plummet wires down the shaft.

The most reliable observation, taken January 8th, 1886, was used until another test was made by Mr. McIntyre, Division Engineer. The readings on the test spikes on January 8th were as follows:

The line on north spike was 1.46 in east of 0, and on the south spike was 0.83 inches west of 0.

Mr. McIntyre, in his later tests, taking the average of the readings, adopted for his line on the north spike 1.29 inches east, and on the south spike 1 inch west of 0. The test spikes were 272 feet apart, and the difference in direction of the former and final lines, for that length, was 0.25, or ¼ of an inch, and at the distance of 2 000 feet from the shaft, which the north heading had then reached, this change made a difference of two inches in the alignment.

At Shafts Nos. 22 and 23 the different alignment tests were more satisfactory. At Shaft No. 22 the following was recorded:

Date.	North Test Spike.	South Test Spike.	Observer.
November 16th, 1885.	0.15 inches east.	0.47 inches east.	W. McC.
January 5th, 1886.	0.00 " "	0.56 " "	W. McC.

These test spikes were 266 feet apart, and the difference in direction for that distance equals 0.24 of an inch. The latest test, being most carefully conducted, was then adopted and carried into the headings. Other tests were made later, but no change made.

At Shaft No. 23 the test spikes were 300 feet apart, and the records of tests on these spikes were as follows:

Date.	North Spike.	South Spike.	Observer.
Nov. 12th, 1885.	0.67 inches west.	0.45 inches west.	McC. and C. F.F.
Jan. 6th, 1886.	0.62 " "	0.47 " "	W. McC.
Difference.....	0.05 inches east.	0.02 inches west.	

This gave a difference in the two tests of 0.07 of an inch at 300 feet, and as the headings from this shaft would be comparatively short—not over 2 100 feet—the difference in alignment at that distance would be only half an inch.

The average of the two observations was taken, and that line established.

Shaft No. 24 was a small shaft, its dimensions being 8 x 11 feet, and was 245 feet deep from the top of the curbing to the invert grade.

Immediately south of the shaft the line of the tunnel curved to the right, to go under the Harlem River, with a curvature of 72 degrees 32 minutes, and radius of 25 feet, and 130 feet north of the shaft the grade changed from 2 feet per 100 feet to go up an incline of 15 feet per 100.

This was a particularly wet shaft, the water pouring down nearly 300 gallons per minute, but by using the illuminated slits, with bulls-eye lanterns behind them, we had no trouble in taking up the alignment.

The test spikes were placed 172 feet north and 30 feet south from the center of the shaft.

The readings on these test spikes were as follows:

Trial lines east or west of 0, on spikes—

	North Head.	South Head.	
December 28th, 1885.	0.25 inches west.	0.60 inches west.	F. W. W.
February 2d, 1886.	0.60 " "	0.33 " "	W. McC.
" 23d, "	0.14 " "	0.51 " "	W. McC.

As the first of these observations made by myself, and the last by Mr. McCulloh, were nearly parallel, the difference in direction being .02 inches in 200 feet, the last one was adopted and the centers corrected to correspond.

The curves in the tunnel, one in the south heading of Shaft No. 20, of 21 degrees 2 minutes 30 seconds total deflection, and one in the south heading of Shaft No. 23 of 7 degrees 11 minutes total deflection; each with a radius of 573 feet, were marked out in the roof of the tunnel, by deflection angles, and measured with a steel tape for chords of 10 feet, and center line plugs, put in at each point as the heading progressed.

The apex of the tangents fell in the side of the tunnel, and by chambering out a little, we were able to set our transit at that point and test the angle by repeated observations, on a test spike and vernier scale set beyond the end of the curve, thus insuring the accuracy of our new course in the heading.

In the south heading of Shaft No. 24 we were required to turn an angle of 72 degrees 32 minutes deflection on a curve with a radius of 25 feet.

As the P. C. came close to the foot of the shaft, with large quantities of water coming down, it was impossible to set the transit there, to run the curve in the usual manner.

We prepared a table of the calculated chords and ordinates from the tangent produced, for each 5 feet around the curve, and measured in for the center-line plugs by this table of ordinates.

By chambering out a little, in the side of the tunnel at the P. I., we tested the angle as in previous cases, and when the P. T. was reached, tested the curve with the transit from that end.

The diagram of curve at Shaft No. 24 will illustrate this work more clearly. (Plate II.)

Before stating what the actual variation in the alignment was found to be when the tunnel headings met, I should mention that the work passed through the hands of three division engineers before completion. Mr. J. B. McIntyre succeeded me on August 15th, 1886; and he was succeeded by Mr. S. F. Morris, as division engineer, in April, 1887, and the work was tested by different parties from those who started it.

In the north heading of Shaft No. 20, as I have stated, the alignment was partially tested through the drill hole, from the surface, at the top of the first incline, about 1 200 feet from the shaft. The variation was then found to be 1¼ inches, but as the drill hole was not plumb, and the points above ground were located by offsets around some buildings, it was uncertain if the error was in the tunnel or above ground, and no change was made.

A slope was afterwards driven into the tunnel, from a point 500 feet farther from the shaft, and the line run down the slope and tested, the difference was then found to be slightly less than one inch.

When the headings of Shafts Nos. 22 and 23 met, on June 22d, 1886, the difference in lines was 1⅜ inches.

And when the headings of Shafts Nos. 23 and 24 met, on November 12th, 1886, the difference in lines was reported by Mr. McIntyre to be but ¼ of one inch.

The headings between Shafts Nos. 20 and 21, and between Shafts Nos. 21 and 22, did not meet until a much later date. I do not know who tested the lines, and have not seen the data for it, but I understand

that Division Engineer Morris reported the error to be less than an inch in either case.

The targets used for fore-sights, above ground, were pine boards 6 feet high and 6 inches wide, painted red and white for each foot high, alternately, on each side of the center, and these targets were permanently secured and braced in their position on the line.

To reach the center-line plugs in the roof of the tunnel, for hanging wires, etc., in a circular tunnel, ordinary ladders were not available, and some tripod ladders were made, which would stand alone in the center of the tunnel. A sketch of these ladders is appended. (Plate V.)

Levels were carried down the shafts by steel tapes, held by a clamp attached to the top curbing of the shafts.

Bench marks in the tunnel were usually made by drilling a hole in a projecting point of solid rock, on the side of the tunnel, above the floor, a wooden plug driven in, and into that a railroad spike, the head of which made an excellent bench.

Cross-sections in Tunnel Excavations.

Taking cross-sections for measuring the areas and quantities of excavation in tunnel work is done on an entirely different system from that ordinarily used in open work above ground.

In tunnel work the cross-sections are usually made by measuring the irregularities of the contour of the section by angles and distances from some point in the vertical plane through the axis of the tunnel, and the elevation of that point (or origin of measurements), with reference to the governing grade line, is determined by leveling.

The instruments generally used for this purpose, so far as I have known, have been rather crude. That used by the writer for measuring the cross-sections of the Weehawken Tunnel was a light pine board, shaped like a semicircular protractor and 4 feet in diameter. Degrees were marked on its circumference for about 200 degrees of angle. This protractor was set on a light wooden easel, and braced to a vertical position, with its center in the axis of the tunnel, and the angles and distances then measured by a rod or tape line.

For the New Aqueduct Tunnel, where sections were to be made every 10 feet for over 30 miles of tunnel, it was necessary to provide some instrument that would be more accurate and more convenient to operate.

The writer devised one, consisting of a circular disk or protractor 3

feet in diameter and attached to a vertical iron rod, the rod sliding up and down in a socket and regulated by a thumb-screw. It was supported by a tripod made of wrought-iron gas pipe, and had extension legs.

The disk could be set in the plane of the axis, and its center raised to coincide with the center of a circular tunnel. Cross-sections could then be taken, and the measurements would show at once if any trimming was needed.

I had one of these instruments made at the Morris Dock office, with the help of a handy man in the engineer corps. As made, it worked well, but was heavy and cumbersome. It could probably have been improved and made much lighter if placed in the hands of a skillful mechanic to manufacture.

Mr. Alfred Craven, Division Engineer of the Aqueduct Commission, and a member of this Society, worked out the problem about the same time, and devised what is popularly known among the Aqueduct engineers as the "Sunflower Instrument" (from its yellow disk of varnished wood). Mr. Craven's plans were sent to Messrs. Heller & Brightly, instrument-makers, who manufactured those used on the New Aqueduct work. The instrument consists of a light wooden tripod with extension legs, a shifting top, ball and socket joint, and leveling screws, similar to a light transit tripod. A vertical brass tube slides through this socket and carries at its top a light wooden circular disk or protractor, about 18 inches in diameter and with its circumference graduated for each degree of the circle, from 0 at the top to 180 degrees at the bottom. (Plates VI. and VII.)

A hard-wood arm or rest, revolving on a central socket, traverses the face of the disk. A wooden measuring rod, about 14 feet long, is laid upon this rest or revolving bracket, in front of the disk, and slid out to touch the rock or surface of the tunnel, the end of the supporting arm at the same time pointing to the degree of angle from the vertical.

The measuring rod tapers from 2 inches to ½ inch in width, and is graduated for feet and tenths from the small end.

The measurements are recorded in the field book like ordinary cross-section notes, the angle read and distance from the center of the disk, taking the place of elevation and distance out.

To insure the cross-section being taken at right angles with the axis of the tunnel, the disk has a small sighting tube attached and per-

Moss Eng. Co. N.Y.

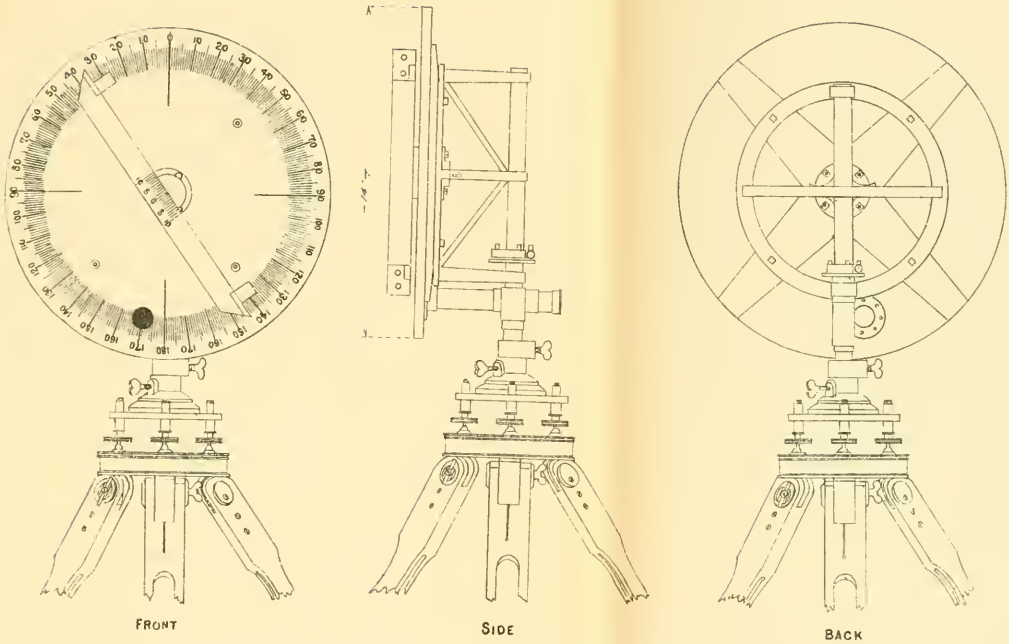


PLATE VII

INSTRUMENT FOR TAKING TUNNEL CROSS SECTIONS

pendicular to its face; when the instrument is set on line and leveled up, a sight is taken through this tube to a distant line light. The elevation of the center of the disk is then taken before the cross-section is measured; about fifteen or twenty measurements will usually cover all prominent irregularities of the contour of the section.

The areas of these cross-sections can either be calculated from the field notes or the plats measured by the polar planimeter. The latter has been found most convenient for this work, and when carefully used, remarkably accurate.

Mr. Craven also devised a circular protractor for platting the disk diagrams in the office, which supplements and carries out the same principle for which the Sunflower disk is used in the tunnel.

It is a circular protractor of German silver, 3 inches in diameter, and graduated from "0" both ways to 180 degrees, and with a transparent horn center, so that it can be placed on the paper and brought to coincide with the elevation of the center of the disk previously marked on the diagram. Four fine needle points on the under side of the protractor keep it in place. A graduated metal arm attached swings around the center and is graduated for feet and tenths on a scale of one-half inch to a foot, on which scale all cross-section plats of the tunnel are made.

This arm marks the degrees of angle, and measures the distance from the center of the points of the cross-section contour.

DISCUSSION.

Edward Wegmann, Jr., M. Am. Soc. C. E.—I have listened with great interest to Mr. Watkins' paper on Tunnel Surveying. Considering the difficulties encountered in this kind of work, the accuracy usually obtained in the "meeting of headings" is very remarkable. On the Fifth Division of the New Croton Aqueduct, of which I have charge, there are ten shafts, the greatest distance between any two consecutive ones being about 2 700 feet. In the nine meetings of headings between these ten shafts the minimum difference of alignment was ¼ inch where the shafts were 2 157 feet apart, and the maximum difference was 1¾ inches, the distance between the shafts being 2 204 feet. This maximum difference occurred, however, where we had been running a reverse curve underground. Some of the facts concerning this particular piece of work may be of interest.

The New Croton Aqueduct, after passing under the Harlem River about 300 feet below high water, turns into Tenth avenue at 179th street.

It continues in tunnel 100 to 150 feet below the surface of this avenue to 152d street, where the old aqueduct comes into Tenth avenue, being, however, near the surface. To avoid blasting directly under the old aqueduct, the tunnel for the new water-supply was turned by means of a reverse curve to the line of Convent avenue, which is parallel with Tenth avenue, and 437 feet to the east. This reverse curve has a radius of 350 feet and a length of 823 feet. Owing to the sharpness of the curve sights of only about 150 feet could be taken from one "set-up," and, consequently, about seven transit-points were required. The whole curve was run from the Convent avenue shaft and the heading was driven about 200 feet farther than the P. T. on the Tenth avenue tangent to the meeting point, where the difference of alignment in the two headings was 1¾ inches. At the Tenth avenue P. T. the difference was only ⅛ inch. There was, therefore, an angle of one to two minutes between the two lines.

The close results obtained generally in tunnel surveying are, in my opinion, not due entirely to phenomenal accuracy on the part of the engineer, nor largely to good luck, but in a great measure to the balancing of small errors to the right and to the left.

The data given by Mr. Watkins show that he never dropped his wires exactly alike, and I suppose that all tunnel engineers have had a similar experience. In deep shafts the wires are never perfectly quiet. In Shaft 25 of the New Aqueduct, which is 426 feet deep, we noticed a very slow, steady oscillation of the wire. In the hurry of the work we did not time these oscillations.

The longest headings which have been run are at the St. Gothard Tunnel in the Alps. The length of this tunnel is 9.06 miles, and as it was worked entirely from two portals, the headings were nearly 5 miles long. The difference of alignment at the meeting point, according to data sent me by a friend who was engaged on this work, was about 1 foot. The difference between the calculated and actual lengths of the tunnel was 15 feet, which error was probably due to difficulties encountered in triangulating over the Alps.

Edward P. North, M. Am. Soc. C. E.—Ought not this oscillation of the long wire to occur once in twenty-four hours—is it not dependent on the motion of the earth? Also, I would like to ask if anybody has used molasses instead of oil or water in the bottom of the bucket?

Charles S. Gowen, M. Am. Soc. C. E.—I believe we tried it once, but it was quite expensive, and we found that the viscous quality of the molasses interfered with the proper vibration and settling of the plumb-bobs. We found in the deep shafts on Division No. 1 that a thin coating of black oil on top of the water in the buckets was better than all oil.

The oil was so thick that it was apt to interfere with the vibration of the bobs in the same way as the molasses, though not quite to the same extent.

We used plumb-bobs weighing 25 pounds, but in deep shafts the slightest interposing force on the wire would interfere with the pull of the plumb-bob.

CHARLES B. BRUSH, M. Am. Soc. C. E.—What instruments, transit and level, were used? Was there any special care in selecting the instruments?

Mr. GOWEN.—We used the ordinary engineers' transit and fluid level.

In shaft work we used ordinary engineers' levels, and part of the time, ordinary engineers' transits.

We had, however, a specially made transit of somewhat greater power than the ordinary transit, and it was used at times. I do not think the results obtained with the larger transit were materially better than those obtained by the use of the ordinary-sized instruments, although it might have been a little easier to work, on account of the large object glass, and consequently more light.

FRANCIS D. FISHER, M. Am. Soc. C. E.—The statement may be of interest that determination of alignment by the use of graduated scales was used in the Hoosac Tunnel.

The scales were securely fastened to wooden plugs driven into holes drilled in the roof of the tunnel. The use of a scale for this purpose was first introduced by myself, and was simply a piece of wood or metal divided into decimals of a foot, along which the plumb line was moved until it was in the correct position, when the reading on the scale was noted. Records of repeated trials of the alignment in this way were made, and from those considered the most reliable an average reading was deduced as being on the correct line. This system was afterwards elaborated by Mr. Wederkinch, Assistant Engineer, who took charge of the alignment work at the central shaft. The instruments designed by him for getting the alignment at the top of the shaft are illustrated on page 69 of "Vose's Manual for Railroad Engineers," Edition of 1874. The central shaft was 1 028 feet deep and elliptical in shape. Plumb lines were suspended down each end of the shaft, giving a base line of about 22 feet from which the alignment was extended into the headings.

The total length of the tunnel was something less than 5 miles. Lines were run in from the east and west portals to a meeting with the central shaft headings, about 1 600 feet east and 2 000 feet west from the shaft. The variation in the lines at the meeting east of the shaft was $\frac{5}{16}$ inch, and west of the shaft, $\frac{9}{16}$ inch. The variation in the levels at the same points was $\frac{9}{100}$ and $\frac{12}{100}$ of a foot.

Mr. GOWEN—I have been requested by several to speak of an experience we had on Division No. 1. Mr. Wegmann alluded to it in his remarks.

At the meeting of the headings between Shafts Nos. 2 and 3, on that division, we thought we would make a practical test of the accuracy of

our lines. These shafts are 6 600 feet apart, and nearly 375 feet in depth. At the time the test was made the tunnels were separated by a breast of rock which remained to be blasted, about 8 feet in thickness. The lines on each side of this breast were run into the heading, and air drills were set up to line and grade, one on each side, and two holes were drilled. Our object was to see if the drills would meet. As a matter of fact, they did meet, and when they were withdrawn a hole was found through the breast about 8 feet in length and drilled practically on a straight line.

F. W. Watkins, M. Am. Soc. C. E.—The question of the vibration of the plumb wires has been spoken of. I can say that so far as I have observed on the shafts in my charge, when everything was quiet and there was no motion or jar from the hoisting machinery, I did not find any vibration, but my shafts were not so deep as some others on the line.

You will notice on the diagram of the vertical slit apparatus that the upper part of the opening is V-shaped and with horizontal lines marked on the face of the brasses.

My idea was that if the wires did vibrate, the vibrations could be observed by the scale on the inclined part of the opening; and by shifting the parts of the V by the thumb-screws, the vibrations across the face could be equalized, and the average obtained for the opening. But I found I did not need to use them (except in the preliminary work), as except when jarred by the motion of the machinery, the wires soon came to a rest, and there was no perceptible vibration.

O. F. Nichols, M. Am. Soc. C. E.—The excellent skill of the engineers of the New Aqueduct has given us, perhaps, the best designed conduit in existence. One great gain to our common experience comes from the opportunity for the application of modern methods to extensive tunneling. It is to be hoped that others of the participants in this great work will follow Mr. Watkins, and that the experience gained will thus be placed within the reach of inquirers so that they may not search in vain, as he did, for information in beginning new work.

The alignment of the Hoosac Tunnel down and from deep shafts has been recorded, and with other examples of the kind has aided many engineers in similar work. In the same way the simple and convenient location of shafts, methods of alignment, etc., followed by Mr. Watkins and his associates, will be of value to many others and go far to simplify the process of tunneling and to remove the dread entertained by many as to undertaking this kind of work. A few years ago we were told that tunneling on Manhattan Island would be much more difficult than in London on account of the advantage which the latter city possesses in having softer material. The influence of the rapid and safe driving of the Aqueduct tunnels is already felt when we read, as in a recent periodical, that a tunnel on Manhattan Island can be more easily driven

than in London because of the existence of the harder material in New York, "well adapted to safe tunneling."

The old method, apparently used to great extent in the Aqueduct, of setting centers in the roof of the tunnels is particularly convenient in the alignment of straight work. It often happens in general experience, however, that the setting of stakes in the roof is inconvenient from the shifting of timbers in soft material and from the difficulty of getting good holes in the right place in friable or uncertain material, and from the difficulty of drilling the holes and setting the instruments under the plugs. This operation is not so difficult in solid material, particularly when the modern method of driving the top heading first is employed. Some stations in the roof are convenient and often essential, but I have found it almost imperative to keep the definite alignment of curves in the bottom, where the line and distance can be conveniently re-run and checked, and for this purpose have frequently set stakes in cement mortar, filling crevices or depressions in the rock bottom to hold the stakes. On a long curve, stations in the bottom at a few hundred feet apart can be made to carry the line with accuracy, while plugs in the roof may be used for approximate alignment by direct sight for straight work and by deflection distances from line of sight for curves.

The "Sunflower" instrument is a simple and ingenious contrivance for securing correct cross-sections of tunnels, particularly where the sections, as in the New Aqueduct, are circular or approximately of the same width and height.

In some narrow-gauge tunnels 16½ feet high and 11 feet wide, with semicircular roof and battened sides, the contractors had kept the top at correct height but too narrow, and it became necessary to determine just how much widening would have to be done on either side at the springing line. A vertical rod was used having a framed right-angled triangle suspended by and made to swing about a pin at the vertex of the triangle and passing through the vertical rod near its top, approximately at the top of the tunnel. The horizontal side of the triangle corresponded in position to the radius of the tunnel arch and was made a little longer than this radius and graduated for measurement from the outer end towards the center line of the tunnel. A long handle served to control the movements of the triangle. When the rod was placed vertically over a station in the bottom of the tunnel and the outer end of the horizontal side of the triangle placed against the side of the tunnel the reading on this horizontal side at the rod indicated the width from the center to the side of the tunnel with sufficient accuracy to determine how much cutting would have to be done at this point.

The cross-section instrument described, and Mr. Watkins' paper on "A Method of Taking Cross Sections in Deep Rock Cuts," recalls an instrument used in cross-sectioning some extensive railway cuttings in Peru, the idea of this instrument itself having been suggested by some

papers in *London Engineering* in 1872–73 on telemeters as applied to determining range for ordnance.

On a tripod with leveling screws a horizontal bar was mounted to revolve in a horizontal plane and also about its own axis. At one end of this bar a telescope was firmly fixed at right angles to it with a graduated circle to determine the angle made by this telescope with the horizon. At the other end of the bar a second telescope was set to revolve in the plane of the bar, and the first telescope with a graduated circle to determine the angle made between the line of sight of this telescope and the bar. The length of the bar between the lines of sight of the two telescopes was just one meter and the bar and the two telescopes would, in any of the ordinary positions, form a right-angled triangle with a horizontal base one meter long.

Suppose the instrument placed at any station in the line of the cutting; the elevation of the station and horizontal bar known; the bar placed in the line of the cutting on straight work or tangent to the curve on curved work; the first telescope, called "A," sighted to any point on the side of the cutting, and the second telescope, called "B," sighted to the same point: the reading of the telescope "B" would give an angle whose tangent would correspond with the distance in meters of the point sighted at from the axis of the instrument, the horizontal bar or base line of the instrument. The field notes for this instrument would be something like this:

Station.	Elevation of Axis of Inst.	Vertical Angle A.	Angle B.	Distance = tan. B. Meters.	Notes.
30	13.20	45° 0′	85° 0′	11.43	.....

Observations having been taken for the different points of change in the cross-section, it could be plotted on cross-section paper and its area determined.

The instrument described was crudely constructed from the parts of two old English transits which had been wrecked in the work and was never extensively used, for the reason that the builder became engrossed in more pressing work where the cross-sectioning of great cuttings did not often occur, and those who succeeded him found the older and cruder methods sufficiently accurate and convenient for their purpose, and were in consequence not much interested in the development of the instrument. To have built a satisfactory instrument for this work would have involved the sending to American or English instrument-makers at a cost in time of perhaps a year's delay. An instrument of this kind would only be valuable under two principal conditions, both of which prevailed on the Peruvian lines.

First.—Where the sections were of peculiar shape and very large, and the points of change often inaccessible, cuttings often occurred whose center depth was over 100 feet, and whose slopes, due to disturbance of

unreliable material, would sometimes run back several hundred feet from the center line.

Second.—Where, as was often the case, there would be but one or two skilled men in the entire party, the assistants being little better than the *peon* labor of the country.

Our New Aqueduct has, in one thing at least, been specially favored in having at all times at easy call an abundance of skilled assistants who could be relied upon to execute difficult work with credit, and these have, I believe, generally had all the time necessary to execute their work with care and accuracy.

Mr. Watkins.—The cross-section instrument described by Mr. Nichols, and used in cross-sectioning some large railway cuttings in Peru, is an excellent example of what may be done in the solving of difficult problems, by the engineer, with a little ingenuity in using the materials at hand. The instrument as described, could be used directly in the plane of the cross-section to be taken, but the base of the triangulation, but 1 metre in length (the distance between the two telescopes), is rather short. The extra labor of carrying additional instruments to the working point was probably the reason of that method being abandoned and cruder means used.

By the method of triangulation, described in my paper on "A Method of Taking Cross-Sections in Deep Rock Cuts" (*Transactions*, June, 1890), no additional instruments were required but the extra attachments of a vertical circle and level tube frequently supplied to an engineer's transit. The range of the vertical swing of the telescope in such transits is too limited to place the instrument in or near the plane of the cross-section to be taken; but by moving 100 feet or more away on the central line, a good measured base can be obtained equal or longer than the distance to be triangulated and all the points in the section brought within the range of the instrument.

A very interesting instrument for measuring the interior of a tunnel, or of a finished masonry conduit, was used during the construction of the Sudbury River Conduit, under the direction of Mr. A. Fteley, Vice-President Am. Soc. C. E., and with his permission I have here re-produced the drawings and description.

In the official report of that work, Mr. Fteley says:

"After the conduit was finished, diagrams showing the measurements of its interior were made at every station, wherever the circumstances of the construction had been such as to demand special attention; these diagrams made with a special apparatus, represent in full size the irregularities of the interior, as it is built, in comparison with the theoretical outline of the section, drawn at a much reduced scale. By this method the irregularities, if any, are made much more conspicuous, and the exaggerated distortion of the outlines defines very clearly the slight movement of the structure and the defects of construction. Some valuable information was obtained from these diagrams.

"As the construction was to cover some years there was ample time to make the improvements suggested by the close study of the portions first built.

"The apparatus used was suggested at first by the necessity of measuring with accuracy the rock section of a tunnel; it was afterwards perfected to be used in the brick structure."

Fig. 1, Plate VIII, represents the inside of the brick sewer or conduit at the section to be measured.

A board, *C D*, is leveled up by the screws, *S*, and firmly wedged across the conduit, so that the point, *O*, is at the theoretical height of the spring line and on the center of the structure.

A light wooden rod, *R*, carries at one end a slot which slides on a pin, fixed at the point *O*. At *P* is a pencil, and at the other end of the rod is fitted a little wheel. On the board, *C D*, is fixed a tablet *A*; on a sheet of paper pinned on the tablet is drawn, at a given scale, the theoretical outline of the section of the brick masonry, with its center at *O*. The rod is then moved by hand in a plane normal to the axis of the conduit, so that the wheel will roll along the brickwork. The pencil, *P*, is so placed that if the brick surface was perfectly correct the pencil would cover the theoretical line drawn on the paper.

"It may be seen that the diagram when completed, will show in *full size on the section drawn at a small scale* all the deviations of the brick surface from the true line.

"The conduit to which this method was applied was 9 feet in width by 7 feet 8 inches high; with a semicircular brick arch supported on side walls of masonry lined with brick; the invert of brick was supported by a bed of concrete.

"The arch was measured as shown before. The irregularities in the curves of the sides and invert (the centers of which were outside of the structure) were measured by a system of ordinates, which by a special arrangement (shown on the figure) for measuring the departure from the theoretical in the direction of the radius, could be very expeditiously taken and recorded on the diagram.

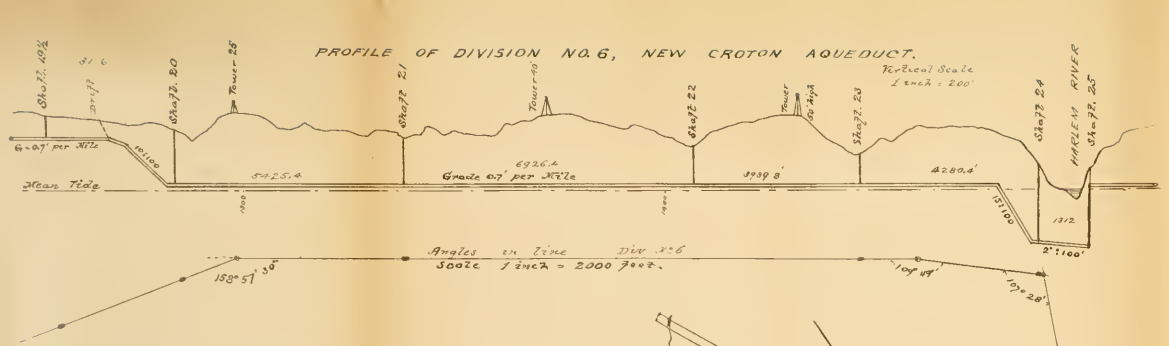
"About sixty sections, a considerable distance apart, could on an average be taken in one day. The diagrams were on a scale of $\frac{1}{20}$."

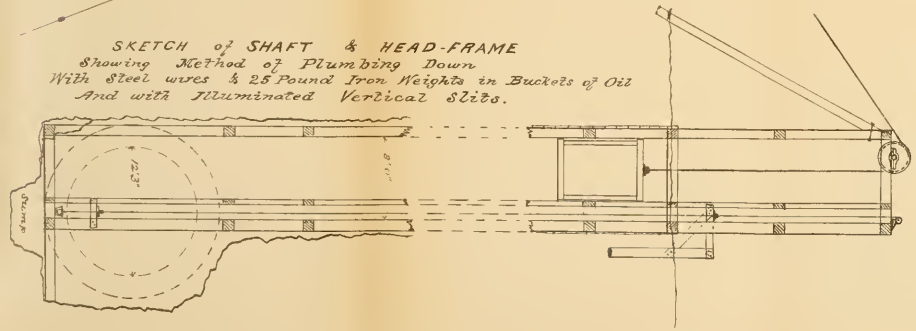
Figure 2 is an example of one of these diagrams where some settlement had occurred, showing the theoretical section on a scale of $\frac{1}{20}$, and the irregularities of the masonry at full size. To the novice in this system of measurements the effect is rather startling, the actual variation being magnified twenty times.

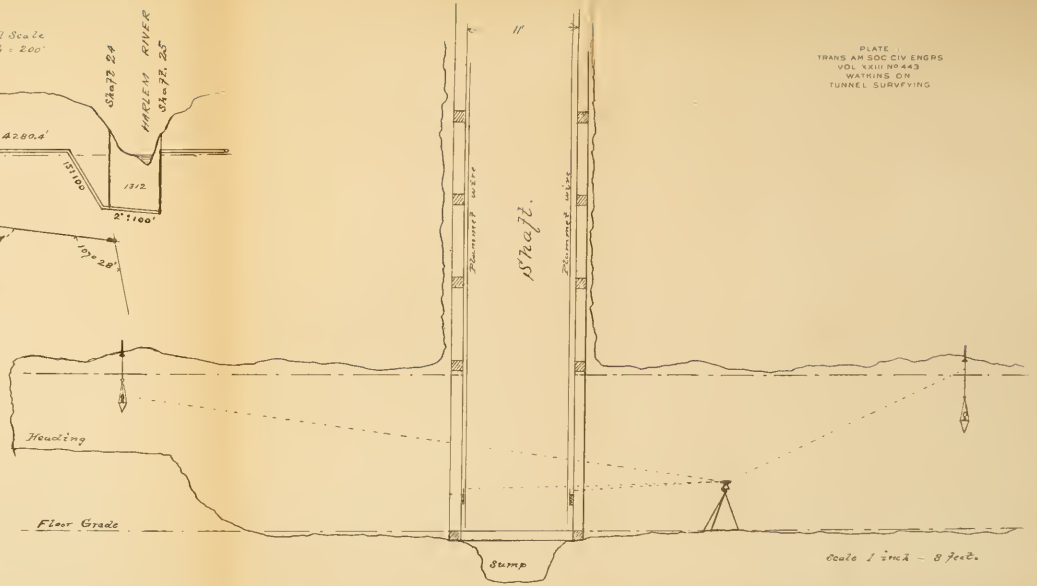
Figure 3 shows the theoretical section and irregularities on the same scale.

A large part of this work was on high embankments or over yielding ground. To those in charge the indications given by the diagrams were most useful, and in some cases they prevented local failures; they not only enabled them to better adapt the design to circumstances, but they gave the means of detecting and preventing the negligence of the builders, and it is thought that in many cases some measurements of this kind would be of great assistance to constructing engineers.

PLATE I
TRANS AM SOC CIV ENGRS
VOL XXIII No 443
WATKINS ON
TUNNEL SURVEYING







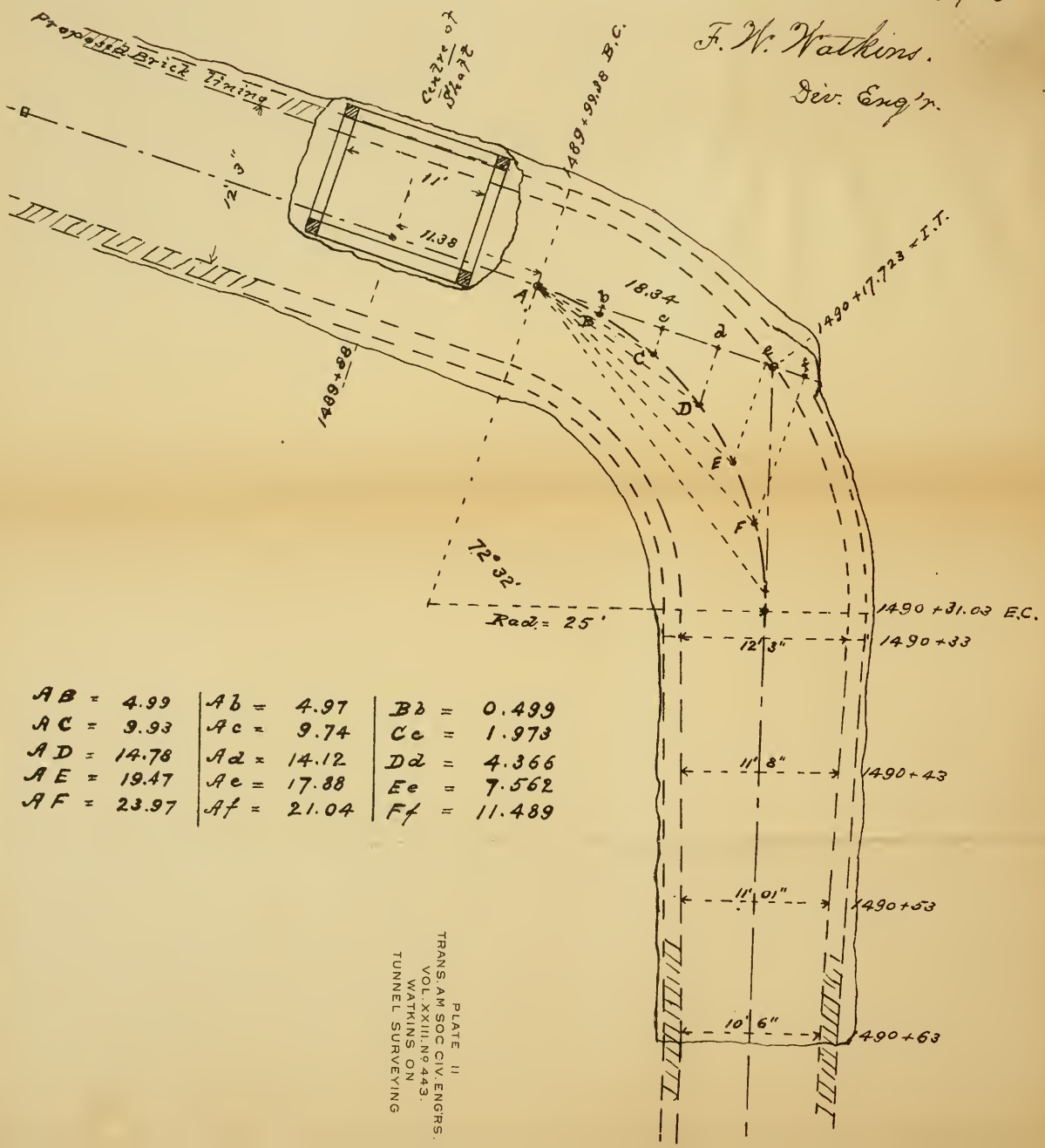
Curve at

SHAFT № 24

Scale 1 inch = 10 feet

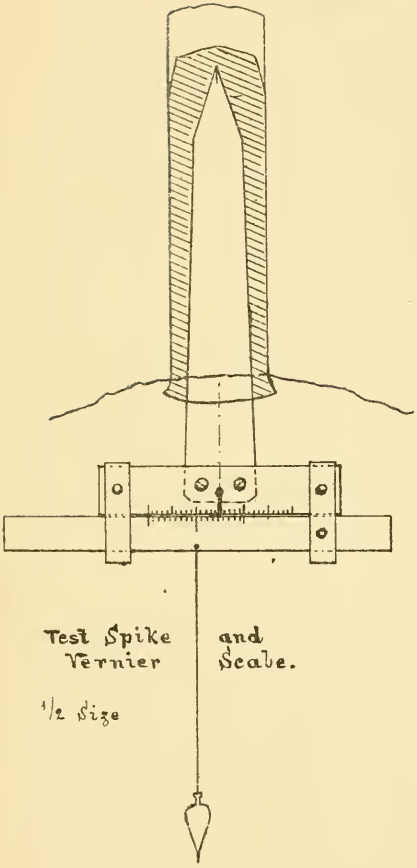
F. W. Watkins.

Div. Eng'r.



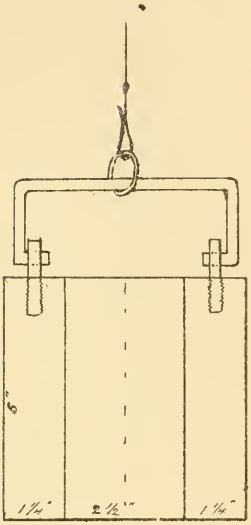
AB =	4.99	Ab =	4.97	Bb =	0.499
AC =	9.93	Ac =	9.74	Cc =	1.973
AD =	14.78	Ad =	14.12	Dd =	4.366
AE =	19.47	Ae =	17.88	Ee =	7.562
AF =	23.97	Af =	21.04	Ff =	11.489

PLATE II
TRANS. AM SOC CIV. ENGRS.
VOL. XXIII. № 443.
WATKINS ON
TUNNEL SURVEYING

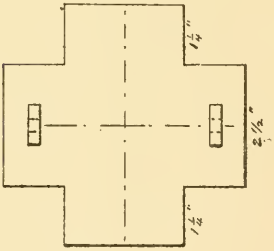


Test Spike and Vernier Scale.

1/2 Size



Plummet Weights
Cast Iron. 25 lbs.

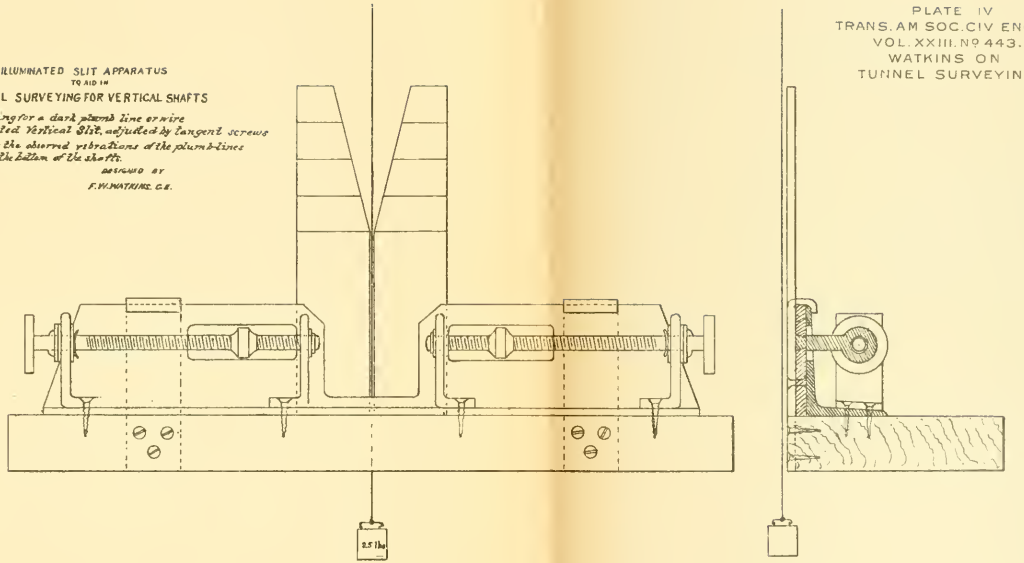


Plan
Scale, 1/2 size.

ILLUMINATED SLIT APPARATUS
TO AID IN
TUNNEL SURVEYING FOR VERTICAL SHAFTS

Substituting for a dark plumb line or wire an illuminated Vertical Slit, adjusted by tangent screws for the observed vibrations of the plumb-lines at the bottom of the shafts.

DESIGNED BY
F. W. WATKINS, C.E.



Sketch of a Tripod, or Tunnel Ladder.
used on Division No. 6.

Scale 1/4 inch = 1 foot.
F. H. Watkins.

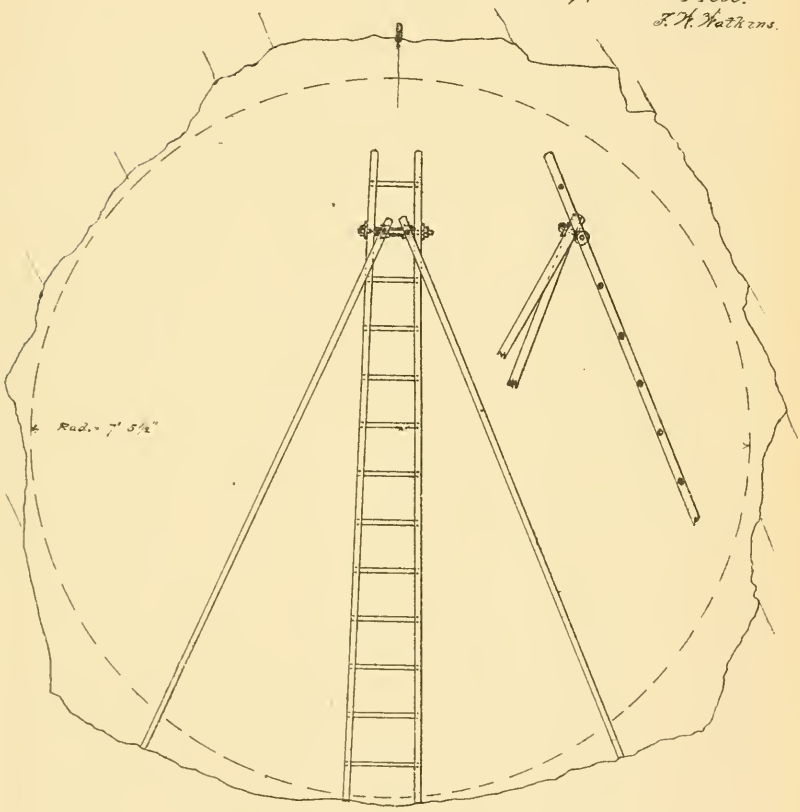


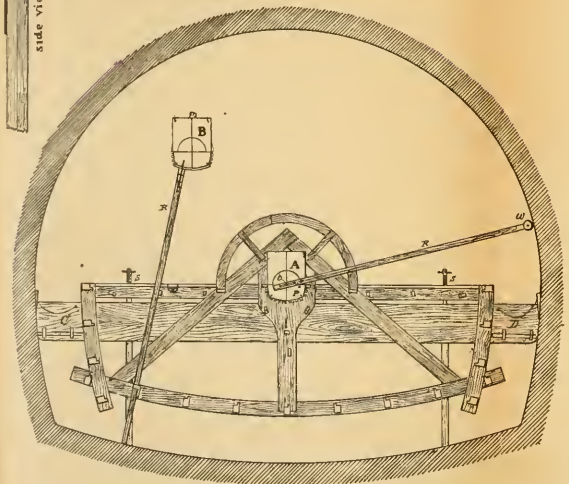
FIG. 1.

APPARATUS FOR TAKING DIAGRAMS
OF THE
SUDBURY RIVER CONDUIT
JUNE 1880.

When taking the arch, the diagram board was placed as shown at A.
When taking the invert and sides, the diagram board was fastened to the end of a rod as shown at B.

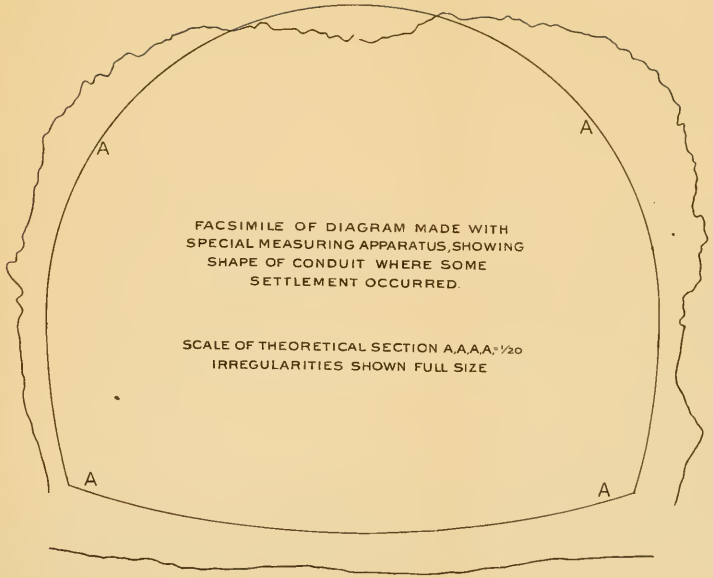
back view of diagram board. ⅛ scale.

side view of rod and diagram board ⅛ scale.



Scale two inches to a Foot.

FIG. 2.



FACSIMILE OF DIAGRAM MADE WITH SPECIAL MEASURING APPARATUS, SHOWING SHAPE OF CONDUIT WHERE SOME SETTLEMENT OCCURRED.

SCALE OF THEORETICAL SECTION A,A,A,A, 1/20
IRREGULARITIES SHOWN FULL SIZE

FIG. 3.

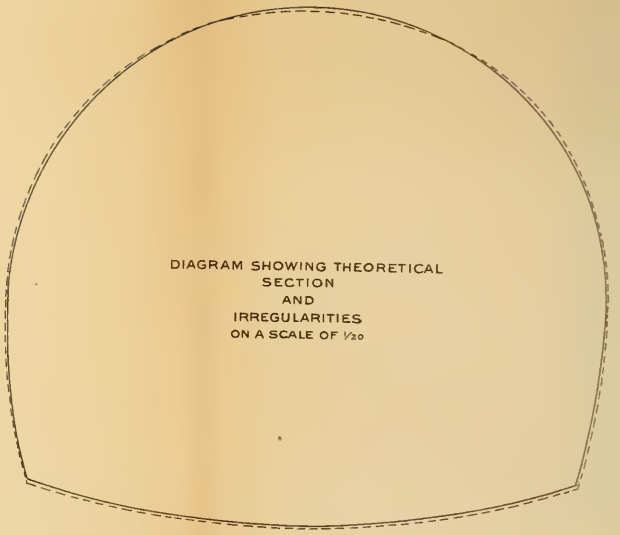


DIAGRAM SHOWING THEORETICAL SECTION AND IRREGULARITIES ON A SCALE OF 1/20

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444.
Vol. XXIII.—July, 1890.

A PRACTICAL METHOD FOR REDUCING THE INTERNAL WASTES OF THE STEAM-ENGINE.

By ROBERT H. THURSTON, M. Am. Soc. C. E.

In the operation of the steam-engine, experiment and observation show that the great losses of heat, steam and fuel which distinguished the real from the ideal engine, and which constitute the greater part of the discrepancy between the computed thermo-dynamic efficiency and the efficiency of the engine in actual working, are due to waste of heat internally by the alternate absorption of heat by the metallic surfaces of the cylinder-heads and piston and the ejection of that heat later to the condenser, if it be a condensing engine, or in non-condensing engines into the atmosphere. Many attempts have been made to reduce this loss, as by rendering the internal surfaces less perfect conductors and absorbers of heat, by superheating the entering steam, and by "compounding" the engine. All of these methods are familiar to the engineer and have been more or less successful. The steam jacket has also been employed, and the internal surfaces have been covered with non-conductors of heat by various inventors; but no device of this kind yet introduced or tested experimentally has been efficient and permanent.

The object of an invention of the writer is to provide an effective non-conducting surface for the internal surfaces of the steam-engine cylinder, formed integrally with the castings of which the engine is composed.

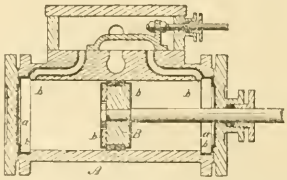
It is well known that ordinary cast-iron is largely graphitic, the carbon contained by the iron amounting to from 2 to 3 per cent. of the whole

up to 5 or 6 per cent., accordingly as the iron has been exposed to a higher or lower temperature in the blast furnace, and as a greater or less amount of fuel is burned in its reduction from the ore. It is also well known that a process of slow oxidation will remove the iron, in part, from the surface so exposed and leave the carbon in a somewhat compact form, which is held in place by a honey-combed mass of unoxidized iron.* This mixture covers the surface of the solid iron to a depth depending upon the method and duration of the action of the oxidizing medium.

One of the best methods of securing the oxidation of the iron is to subject it to a dilute acid—say, for example, a solution of sulphuric acid in water in the proportion of one part of acid to ten or more of water. The writer has found that a piece of iron submitted to a very weak solution of acid for ten days produced good results, the graphitic surface being fairly dense and compact.

The idea is to expose the internal parts of the engine (such as the heads of the cylinder, the two sides of the piston, the internal surfaces of the ports, and the clearance space) to the action of such a solvent for a suitable length of time, thus converting those surfaces into highly carbonized material, which is a comparatively poor conductor of, and has a low capacity for, heat. It is impracticable to so treat the rubbing surfaces; but, as they are kept in a high state of polish, their capacity for taking up heat is thus greatly lessened, and it is therefore not so necessary to treat the surfaces subjected to friction.

In modern engines, especially those of the "high-speed" type, the cylinders are of large diameter and small length, and the stroke of the piston is short, thus exposing large areas of piston and cylinder-head, which are effective in condensing the steam and reducing the efficiency of the engine. These large areas I propose to treat in the manner described.



Referring to the drawing, the interior surfaces of the heads *a* and the interior surfaces of the ports and passages of the cylinder *A* are pro-

* Shown by analyses made by the writer as early as 1859.

vided with a coating, *b*, of graphite produced in the manner above described. Opposite sides of the piston, *B*, are treated in like manner.

In addition to rendering the surfaces graphitic in the manner described, we may fill the spongy surfaces so prepared with a substance which is a non-conductor of heat—such as drying oils, shellac, or other adhesive non-conducting materials—which will decrease the conducting and heat-storing properties of the surfaces, and at the same time aid in giving them permanence in the presence of variations of temperature, pressure and humidity, such as inevitably occur in all engines.

The effectiveness of this plan, as devised by the writer, was studied, and its influence on the efficacy of the engine was investigated, at his request, by Mr. P. M. Chamberlain, in the Sibley College laboratories, Cornell University, in the early part of the present year.

An engine built especially for the laboratory and for experimental work by the Messrs. Payne, of Elmira, N. Y., was employed, and the process consisted in measuring the flow of heat through its head into a calibrated calorimeter. Several series of trials were made at various times, and under differing conditions, each usually lasting forty minutes.

In the first trial a head was made for the steam cylinder of what is familiar in the foundries of Western New York under the name of "Tuscarora" iron. Its thickness was one inch. The outer face was smoothly polished, as is usual; the inside was simply smoothly faced-off in the lathe with the ordinary finishing cut. The inner face was tested under conditions described as follows:

No. Test.	Condition of Inner Surface.
1.	As finished; greasy.
2.	As finished; greasy.
3., 4., 5.	Same; washed clean with benzine and dried.
6., 7.	Oiled with lubricating oil.
8., 9.	Washed clean with benzine.
10., 11., 12.	After exposure to nitric acid sixteen hours; then oiled with linseed oil.
13., 14.	Similarly treated with hydrochloric acid, and oiled after twelve hours' exposure.
15., 16.	Sulphuric acid, 1; water, 2; for forty-eight hours; then oiled, and allowed to dry twenty-four hours.

Rankine's formula was employed in making comparisons : *

$$q = \frac{t' - t}{\sigma' + \sigma + \rho x};$$

in which t' and t are the temperatures of fluid on each side the plate, σ' and σ are the resistances to heat-transmission through those surfaces, and ρ is the co-efficient of internal thermal resistance, and which may be neglected as insensible here. The equation gives q the quantity of heat, in $B\ T\ U$, transmitted per square foot per hour.

Let Q = total heat transmitted, then for the case in hand,

$$q = \frac{3}{2}\,\frac{Q}{0.171} = 8.772\ Q$$

The value of Q varied from about 5 550 $B\ T\ U$ in cases 1 and 2 to 3 000 and 3 100 in cases 15 and 16, as shown in the table which follows:

TABLE.

No. Test.	Q.	$t' - t$.	$\sigma' + \sigma + \rho x$.
1	5550.9	236.0	0.004844
2	5566.4	238.0	4874
3	5444.2	238.8	4854
4	5139.3	239.1	5303
5	5055.8	239.8	5410
6	5523.9	236.4	4878
7	5444.3	236.9	4962
8	5620.4	233.7	4740
9	5515.6	234.7	4851
10	5101.9	236.6	5286
11	5439.7	235.7	4950
12	5581.8	235.6	4812
13	5347.5	234.3	4985
14	5741.1	233.5	4635
15	3020.4	225.3	8279
16	3104.9	225.5	8503

In the first four runs the gradual increase in resistance noted was attributed to the gathering of rust on the exterior surface, no oil being used to protect it from the water, and to the gradual loss of greasiness on the interior. In Nos. 3, 4, 5 no sensible change appears other than the gradual alteration just noted ; nor is any notable result attained until the last pair of data are reached. It then becomes evident that treatment with drying oil, even after so short a period of exposure to acid and so

* Steam Engine, p. 258.

short a time allowed for drying, has an exceedingly important effect in reducing conductivity and preventing heat storage and transmission.

By even a superficial treatment with acid and oil, the saving of heat is 40 per cent.

$$\frac{8500 - 5000}{8500} = 0.40, \text{ nearly,}$$

and this, it would seem, should measure the proportion by which internal cylinder condensation is diminished. With more prolonged treatment with acid and a more efficient absorption of oil and more complete drying, the gain would be considerably greater than here appears, and as will be hereafter shown. The writer has estimated that this simple treatment may reduce cylinder-waste by at least half; which, in ordinary practice, means from 10 to 20 per cent. of all steam used, the figure being greatest with smallest sizes of engine.

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(Vol. XXIII.—July, 1890.)

THE DEWITT CLINTON.

By G. H. THOMSON, M. Am. Soc. C. E.

The locomotive "DeWitt Clinton" was the third engine built by the West Point Foundry Association, of New York City.

The first engine (the "Best Friend") exploded; the second engine was a failure; the DeWitt Clinton was a success.

The first passenger train of America was drawn by the DeWitt Clinton, August 31st, 1831, over the Mohawk and Hudson Railroad (now the New York Central and Hudson River Railroad) from Lydia Street Depot, Albany, N. Y., to Schenectady, 17 miles distant.

At Albany the ascent from the Hudson River to Lydia street was made by inclined plane. The DeWitt Clinton left Lydia street in good condition and continued the up-grade trip to a point half-way between Albany and Schenectady, when she stopped. Here the line, which had thus far been straight, was about to deflect three-quarters of a degree, and the angle being made at once (curves not then being known) she stopped,

and having turned the three-quarters of a degree, proceeded on to Schenectady, where the train safely arrived, having made the journey in about one hour.

Before the train left Lydia street a Mr. Brown (an artist in silhouette) made a picture of the train, including passengers; this sketch was deposited in the Connecticut Historical Society, and about fifteen years ago was published, and is known as the Howard drawing. The outline of the DeWitt Clinton in this picture is good and also those of the cars, but there have been added to the published picture two misstatements which should be corrected for history's sake. The first error is in the name of the engine, there given as the John Bull. It should be stated here that the engine John Bull did not resemble the DeWitt Clinton. The John Bull was a heavier engine, and had been selected to make the run to Schenectady, but not arriving from Europe in time, the DeWitt Clinton was sent to Albany and did pull the first passenger train, as above stated.

The second error is the statement that the engineer that pulled this first train was John Hampson, an Englishman. The name of the engineer was David Matthews, a man still alive, and somewhere near ninety years old.

The following letter explains itself:

"1161 Howard Street,
San Francisco, Cal. July 21st, 1885.

"'G. H. Thomson,
Civil Engineer Room 21 *Grand Central Depot*,
New York City."

"My Dear Sir:"

"Yours of July 12th is to hand with the Photograph of My Old Locomotive Engine DeWitt Clinton Which I highly appreciate as it is a rare production in comparing it with My Drawings I find sum thing that does not come up to the point but they are errors in the drawing that the Steam doom is to high and the Smoke pipe is to large in diameter and the shape at the botem is not right, but it is as correct as most drawing of such difficult pieces of Machinery, I made a drawing of her when I placed her on the road which is Fifty four years past the last June and this time fifty four years ago I was harnising up the Young Iron *horse*, that Weighed only three and Half ton. that was the limited weight for her and shee was Made of the best Material and braught down to the finest proportions and Finish such as never Since been done you will See on examining the pair of *wheels* that is left of hers, that they are a curiosity in themselves and are *What* I *call*, a *reglar Gerder Moving Sus-*

pension Bridge each arme is a road that suspends the load from the top corde and is brasing from the hub *to the* perifery Each arme is duftailed in the fella, and ground so that it has perfect fit and is Supported by a nut in the hub theas arms wear all bright and was cep so that they through thear shadow—when the Sun Struck them that give a butiful sight, I was at this time Fifty four years ago Preparing this DeWitt Clinton for the corce I left my Fathers & Mothers House on the twenty fifth day of June 1831 that was North More St. New York City and was Experimenting and fitting up tender Cover &c training her to the track till the 9th. Day August 1831 When I took the Colt and gave the old boys a Ride be hind him over the hills to the Mohawk "

"Respectfully &c."

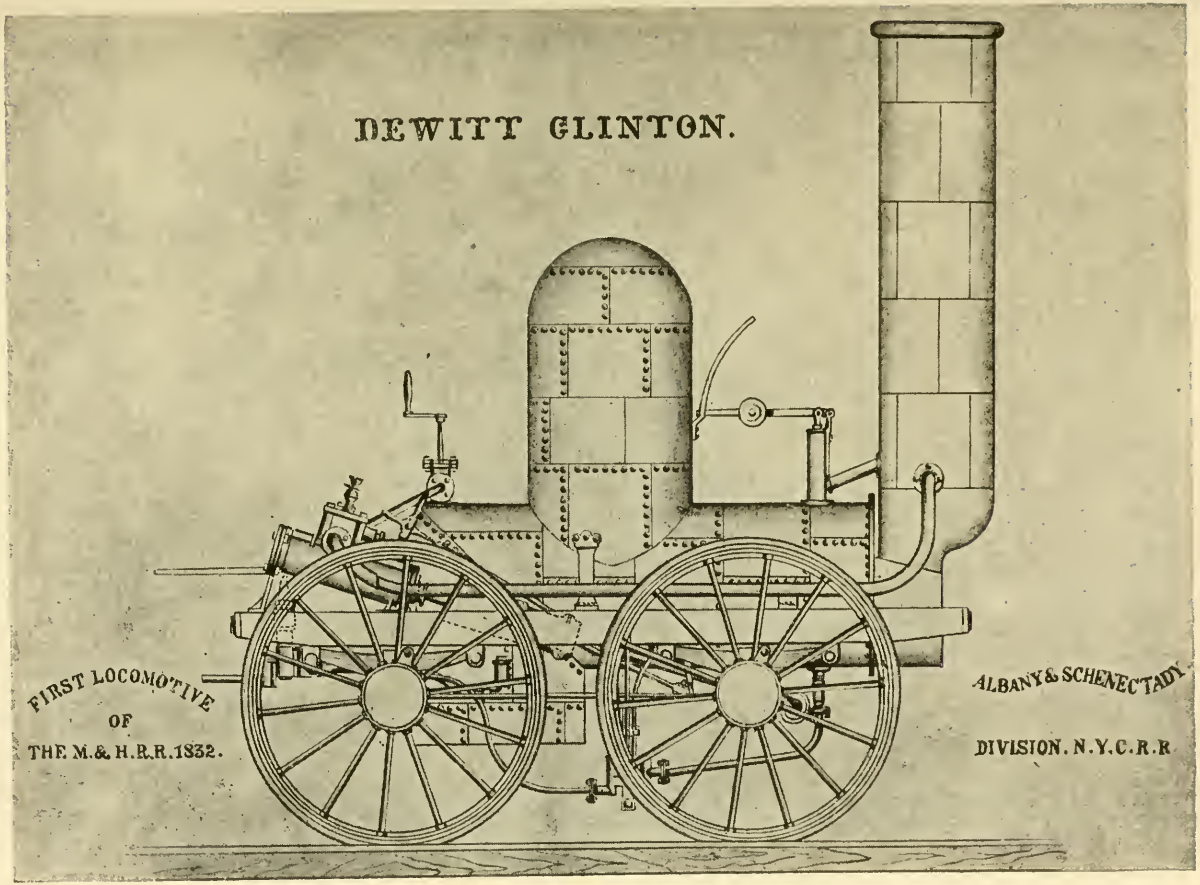
(Signed)

DAVID MATTHEWS.

The DeWitt Clinton was dismantled in 1848. One cylinder went to West Troy and became part of a stationary engine; the other was said to have been used on a small ferry-boat. One pair of driving-wheels is at present at the Grand Central Depot, New York, and attests the close scaling of the drawing from which this photograph was made.

The drawing of the "DeWitt Clinton," on a scale 1 inch = 1 foot, was executed by L. P. Hannas; the lines are clear and shading neatly done. The exact date when the drawing was made is unknown, but can be shown to have been prior to 1840, and to have been made from measurements. The lettering, "First locomotive of the M. & H. R. R., 1832," and "Albany & Schenectady Division, N. Y. C. R. R.," was added between the years 1853 and 1856.

The drawing of the DeWitt Clinton (see Plate IX), now in the Society rooms at New York, is not the engine as built; the wheels there shown are like the actual wheels, but the number of spokes is shown as one too many, and the boiler and some other details were not built after this design. Possibly this drawing might have been the first study for this engine.


DEWITT CLINTON.
FIRST LOCOMOTIVE
OF
THE M.& H.R.R.1832.
ALBANY & SCHENECTADY
DIVISION. N.Y.C.R.R

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXIII.—August, 1890.)

THE RIVER SPANS OF THE CINCINNATI AND COVINGTON ELEVATED RAILWAY, TRANSFER AND BRIDGE COMPANY.

By WILLIAM H. BURR, M. Am. Soc. C. E.

WITH DISCUSSION.

The structure which forms the subject of this paper crosses the Ohio River at Cincinnati, Ohio, and with its approaches forms a part of the Chesapeake and Ohio Railroad system. It acquires its interest as a piece of engineering chiefly from the magnitude of the individual spans of which it is composed. There were no special engineering difficulties to be overcome either in the substructure or superstructure, but the central span of the three, 550 feet long between centers of piers, and 84 feet deep between centers of chords, is the greatest simple non-continuous truss span yet constructed. The two spans which flank the center or main channel span are 490 feet each between pier centers, with center depths of 75 feet; and the fact that all the spans carry a double track railway with two roadways and two sidewalks, renders them also the heaviest non-continuous trusses which have yet been built either in

this country or in Europe. The detail drawings accompanying this paper show all the main features of the trusses and floor systems and their connections which are of any special interest. As they indicate, all the main parts of the trusses are of steel, while the lateral and transverse systems of bracing and the floor-beams and stringers are of wrought-iron.

With the exception of the connection between the floor-beams and posts, and the web system, there will be found few features not ordinarily included in the best American practice for heavy spans. All connections are central, and so designed as to eliminate essentially all secondary stresses. The system of web members used, and which has been developed by the Phœnix Bridge Company for its long spans, is seen to be single, and it is of interest in passing to note that if a single system of bracing may be used for trusses of the dimensions and weight of these under consideration, there would seem to be no case where it may not be advantageously employed. There is thus avoided all the ambiguity and secondary stresses which are inevitable to a greater or less degree when any multiple system of web members is used.

The boring of individual truss members was done with such lengths as would eliminate all secondary stresses whatever at a condition of loading intermediate between no moving load and a full moving load. As the latter condition of loading will very rarely occur, these normal lengths will reduce the secondary stresses to an absolute minimum; in fact, will reduce them to such small magnitude as to leave them with no importance whatever. The connection between the floor-beams and posts, which is made by means of close-fitting turned bolts in holes drilled with those members assembled, is of such a character as to secure all the advantages of a rigid connection and at the same time eliminate all tension upon the connecting bolts, leaving them to transfer shear only; at the same time the weights of both railway and highway floor systems are transferred centrally, so as to bring an equal distribution of weights upon all of the web members intersecting at any lower chord panel point. Under the requirements of the specifications all rivet holes in the plates and angles forming the upper chords and end posts, and nearly all intermediate posts, were made with multiple drills of six drills in a gang. The only exceptions to this statement were some light plates and angles in a few of the intermediate posts, which were punched and reamed.

Much difficulty was experienced in obtaining metal for the heavy plate links at the upper ends of the end posts which would fill the requirements of the specifications, or sufficiently near thereto. A number of steel plate makers felt confident of being able to produce such thick and heavy plates as would meet the requirements of this case, but repeated trials were failures. The metal would be very low in elastic limit as well as in ultimate, and develop porous places in the interior of the mass. The whole difficulty lay in the small amount of work which was put upon the metal between the ingot and finished plate. Messrs. Graff, Bennett & Co. finally produced a number of plates of open hearth steel which met the requirements of the specifications. Their financial difficulties coming on at this time, however, prevented their completion of more than a few only of the plates required. The remaining plates for these heavy links were made of Bessemer steel and produced at the Homestead Mills of Messrs. Carnegie, Phipps & Co. The experience with these plates was very interesting in itself, although the difficulties encountered threatened at one time to result in a serious delay to the progress of the work. It demonstrated in a peculiarly clear and effective manner the improvement in the quality of the metal produced by an increased amount of work. The most porous portions of several plates were a number of times worked down under a hammer to bars of most excellent steel, alike in respect to its elastic limit, ultimate resistance and ductility.

The 7-inch steel eye-bars were forged from open hearth steel, while the 8-inch bars were forged from Bessemer steel.

The steel pins were forged from open hearth metal.

Before proceeding with the actual shop work on these spans, many careful tests on the effect of the various shop manipulations of the steel material were made in order that the greatest confidence might be placed in the resulting work. Rivets both with counter-sunk and full heads on one and both sides of plates were driven, and the hammering continued throughout the stage of blue heat as the metal cooled down; heads were then knocked off, or the counter-sunk rivets driven out in such a way as to give their material as much abuse as possible. The results of these tests were in every way highly satisfactory and showed that the material selected was admirably adapted to its purpose. They also revealed the fact that with proper material in steel rivets, that is, with phosphorus not over about five hundredths of

one per cent., and with an ultimate resistance of about 60 000, they can be used so as to stand more abuse than those of iron and give far more strength and toughness.

Steel plates were also sledged with heavy hammers and stretched at various points irregularly until they were badly curved out of shape and left curved with hammer marks; after this maltreatment specimens were cut out and tested in tension and bending. The ductility in such cases was found to be only a little injured, with an ultimate and elastic limit somewhat raised. Some of these tests were reported in the *Transactions* of this Society for October, 1887, Vol. XVII, p. 185. The results of these special tests and the general result of the whole work demonstrated the fact that the growing confidence which has been placed in structural steel is not misplaced, but most firmly founded. The various tables of results appended to this paper are extracted from the various tests which were made to establish the character of the material in the general progress of the work. They are true representations in all cases of the average results obtained. The entire record of tests has not been incorporated in this paper for the reason that a very large number are shown, and the whole would have added unwieldy bulk to it without conveying any additional information.

The specifications under which this work was executed were drawn up and proposed by The Phœnix Bridge Company, by whom the entire work of these spans was designed, constructed and erected, and accepted by the Covington and Cincinnati Elevated Railway, Transfer and Bridge Company.

The actual shop work of the construction of these spans was begun in March, 1888, and the last of the three spans was coupled and traffic passed on the 25th of December of the same year.

The weight of iron and steel in the two 490 feet and one 550 feet spans is almost exactly 10 000 000 pounds. The length of the Covington approach is about 1 533 feet, while that of the Cincinnati approach, including the structure carrying the many diverging tracks to the freight depot, is nearly 2 300 feet, making a total length of structure carrying a double track railway, double roadway and sidewalks of almost exactly one mile. The total weight of metal in this entire structure is 20 360 000 pounds.

The work of erection of these spans under ordinary circumstances involved no special difficulties, but the abnormal character of the season

demonstrated with great intensity the full dangers which may beset any work carried on in the Ohio River. In June, 1888, as soon as the stage of the water would permit, false work was begun at the Covington end of the south 490 feet span. In the early part of August this span was safely coupled, after a most dangerous flood in the river during the latter part of July. This flood caused a rise in the river of about 25 feet; large quantities of drift were brought down by the current and lodged against the false work then carrying the large traveler and about two-thirds of the trusses and railway floor. The pressure of this mass was so great that it pushed the false work about one foot down stream, but did no other damage. Incessant rains made it impossible to commence putting the iron of the 550 feet span on the falsework until about the middle of August. Shortly after that time the river experienced another flood rise of 27 feet with enormous quantities of drift. These conditions prevented much progress in the work, but on August 26th about 700 000 pounds of iron and steel railway floor and eye-bars were placed on the false work, which at that time was not quite completed for the 550 feet span. For several days previous to August 26th it became evident that the high water was becoming exceedingly dangerous, and the entire erection force was occupied in attempts to protect the work that had already been done and remove so far as possible the drift which was continually coming down the river. But on that date (August 26th), with some 36 feet of water in the river, the drift formed a continuous mass for over 500 feet up-stream from the bridge, and in spite of expensive temporary protection it swept the false work, the large traveler and nearly 700 000 pounds of iron and steel work down the river in an absolutely complete wreck. Masses of this wreckage were stranded down the river from the bridge site for fully 50 miles, and two steel eye-bars were picked up 45 miles from the bridge. Subsequent careful examinations of the river bottom showed that the false work piling failed by a very little only, in holding its own safely against the flood and drift. The bottom was found to be scoured out for a considerable portion of the span to an increased depth of 10 to 12 feet, thus removing from the false work piles the support which they required. This scour was, of course, due to the large mass of drift which had become nearly solid from the water surface to the bottom of the river on the up-stream side of the piling. Had the bottom not scoured, the false work structure would have safely withstood the flood ; in fact it held

firmly for a number of hours with the flood at about its maximum height, and the water commenced to recede within six hours after the failure. Contrary to usual experience the river maintained its high water during the entire month of September and the early part of October, although it receded for short periods, at several intervals, from 5 to 17 feet.

The Phœnix Bridge Company at once ordered new lumber for false work, traveler and piles at as many points as possible in Ohio, Indiana and Georgia, so as to insure the concentration of the largest quantity in the shortest time. New hoisting apparatus also had to be ordered and an extensive electric light plant was founded and started within four days after the wreck, and the entire bridge site was thus illuminated and all the operations of pile-driving, placing false work and erection of the iron and steel work was actively continued both day and night; in fact, from the day of the wreck, on August 26th, to the completion of the structure on December 25th, there was no cessation of operations either night or day. It became evident from the phenomenal character of the season that the usual autumn low water in the Ohio River was not to be experienced, hence in order to give the new false work of the 550 feet span thorough protection, two lines of heavy piling were run obliquely up-stream about 600 feet from each of its extremities, thus forming by their intersection a V-shaped protection, with the angle of the V about 550 feet up-stream. Each of these lines was formed by piles 5 feet apart centers, backed by a group of six piles every 40 feet. The two lines were then sheathed by 4 x 6 scantling. A depth of water 45 feet in the river would have just submerged this protection, but it was considered safe to neglect the expectation of such a rise, and subsequent events justified the anticipation. This protection was found to act most admirably and formed a complete safeguard to the false work against a number of rises in the river with very considerable amounts of drift.

So actively was the work prosecuted that just five weeks from the day of the wreck the entire false work, including piles, the two travelers and about 1 200 lineal feet of pile protection, were completed in place, the iron and steel floor once again being placed on the false work. During this time over 950 piles had been driven and nearly a million feet of lumber framed and placed in the false work and the two travelers; the traveler for the handling of the iron and steel being as

large as any ever constructed. The 700 000 pounds of iron and steel railway floor had within the same time also been entirely reconstructed from new material at the Phœnixville shops and delivered at Cincinnati. The erection of the iron and steel work of the 550-feet span was then pushed forward night and day, and was entirely completed, including the floor and all lateral and transverse bracing, on the 28th of October, and it was swung clear from the false work immediately thereafter. The coupling of this span was completed just after a heavy storm with a flood rise of 27 feet, making the third period of high water since the beginning of the work. This flood rise continued at or near its high water mark for some three weeks, and prevented the driving of any piles for that length of time in the north 490-feet span. During the last of November, however, the water commenced to recede and the remaining pile driving and false work were soon completed. On December 9th the first iron work for the railway floor was run out on the false work of the north 490-feet span at the Cincinnati end of the structure, and the erection of the remaining iron and steel work was carried on continuously day and night until the last coupling was effected, as stated, on December 25th. The placing and erection of all the iron and steel work of this span, including railway floors and all lateral and transverse bracing, was completed in sixteen days, on the last of which the first regular railway traffic passed over the bridge, from which time schedule trains were regularly run.

Substructure.*

The shore piers of the two 490-feet spans rest on piles capped transversly of pier with 12 x 12-inch white oak timbers, which in turn carry longitudinally of pier nine lines of the same 12 x 12-inch timbers. These latter carry a solid 12-inch white oak floor or platform about 72 x 36 feet, on which the masonry is placed. The piles are placed 4 feet apart, centers in both directions. They are white oak sticks driven to refusal 30 to 42 feet into the clay and gravel of the banks. There are five bottom courses of masonry, each 27 inches thick and each stepped off 12 inches. The masonry of the main body of the pier surmounts these bottom courses with the batter and dimensions shown on Plate XII.

* The writer is chiefly indebted to Mr. Epes Randolph, Chief Engineer of the Covington and Cincinnati Elevated Railway, Transfer and Bridge Company, for the ample notes on which the following account of the substructure work (frequently in the words of Mr. Randolph) is based, although Mr. Charles Sooysmith, M. Am. Soc. C. E., has also given him valuable assistance in several particulars.

The 24-inch sub-coping courses on all the piers are of Kentucky freestone; while all the 24-inch coping courses are oolitic limestone from Salem, Indiana. This latter is a very compact stone and offers a compressive resistance of about 12 000 pounds per square inch; its ratio of absorption does not exceed 2 per cent. of its weight. The belting courses are of a very superior sandstone from the interior of Kentucky, known as Kentucky freestone. It possesses a compressive resistance of about 15 000 pounds per square inch and a ratio of absorption of 3 per cent. The Kentucky shore pier was built of this freestone throughout, while the Ohio shore pier is entirely built of Ohio River freestone. The two river piers are faced with Greensburgh limestone, and both are backed with Ohio River freestone from top of caisson to belting course. Above the latter the same backing was used in one river pier and the Kentucky freestone in the other.

The two river piers, one at each end of the 550-feet span, rest on pneumatic caissons 81 feet 3 inches x 34 feet 10 inches in plan at cutting edges. The batter of the sides of the caissons is 1 in 15. The walls of the working chamber for a distance of 6 feet below the roof are 4 feet thick and composed of three shells of 12 x 12-inch sticks with four of 3-inch sheathing, arranged alternately. The outer shell of 12-inch sticks is carried down 2 feet below the interior and 1 foot below the center one; the former carries at its lower extremity a 6 x 9-inch piece of oak chamfered to form the cutting edge. The distance from base of cutting edge or shoe to the roof of the caisson is 8 feet 9 inches. The outside 3-inch sheathing is laid on vertically and well caulked. The inside shell of 3-inch sheathing is also laid on vertically, but the two intermediate shells of the same material are laid diagonally, thus binding each entire wall together in one solid unit. The roof of the working chamber is also covered with 3-inch pine sheathing, and the walls of the working chambers are braced longitudinally to the caisson by two pieces of 12 x 12-inch oak sticks running the entire length and tied securely to the end walls. These longitudinal sticks are held at the proper distance from the roof of the caisson by 12 x 12-inch oak sticks and 1-inch wrought-iron rods. The transverse bracing of the walls of the working chamber are held rigidly in place by five 14 x 14-inch oak sticks and 2-inch round wrought-iron rods running through the walls to the outside sheathing. The roof of the caisson is formed by seven solid transverse and longitudinal layers of 12 x 12-inch pine sticks.

Above the caisson is constructed the crib work, or grillage, formed of alternate series of four longitudinal and eight transverse 12 x 12-inch pine sticks, with interstices, forming by far the larger part of the mass filled with the best of concrete. This crib work consists of thirty-five layers in the Ohio caisson and thirty-four layers in the Kentucky one, above which comes the masonry of the pier proper. As the building of the crib progressed, the work of concreting was completed after about every three courses of the timber work were finished. The top of the crib work is about 30 x 76 feet, and the distance from the top of the crib work to the cutting edge is 52 feet 5 inches for the Ohio and 51 feet 3 inches for the Kentucky caisson.

Plate XIII shows in complete detail the method of constructing these caissons and their dimensions as well as those of various other parts. The Ohio caisson and crib contains about 527 000 feet B. M. of timber, and the Kentucky caisson and crib about 514 000 feet. The timber in the caisson and cribs was in the main drift-bolted, although some parts of the working chamber were joint-bolted, while the 3-inch plank was spiked.

The composition of the concrete used in the crib work was as follows: One part Louisville cement, one part clean sand, and three parts stone broken to pass through a 3-inch ring, all well mixed, with just sufficient water in the sand and cement to give a thick paste. A No. 3 Gates crusher with a 15-horse-power engine was used to crush the stone. The capacity of the crusher was about 80 cubic yards per day of ten hours, and the force used in this part of the work consisted of twelve men, one engineer, three men and helper, three men at chute, five men delivering stone to the crusher barge. The chute at which the three men were employed was the channel along which the broken stone was delivered from the crusher to the barge, on which it was carried to the pier in process of construction. The concrete mixer consisted of a Sooysmith proportion meter, screw conveyor and mixer, worked by a 6-horse-power boiler. The usual total force employed in and about the mixer was about twenty-five men, while the concrete force in the crib usually consisted of about twelve men.

The Ohio caisson was launched June 2d, 1887, and was kept afloat by a false bottom, which had been placed temporarily just below the oak stiffening timbers, which braced the walls of the working chamber; it was made of 3-inch plank. Immediately after the caisson was launched

an air compressor barge was lashed to it, connections made and air used to aid the false bottom to float the caisson. When launched June 2d, the caisson working chamber had only one course of the deck laid above it. From June 2d until June 20th the time was spent in completing the roof or deck of the caisson, after which it was floated into position behind a row of protection or guide piles V-shaped, with the angle upstream. The Ohio caisson was settled to river bottom June 20th and rested on a level surface 7 feet below low water, and the false bottom was entirely removed July 1st by sawing into small pieces and shoving the portions under the cutting edge.

The pneumatic machinery and electric light plant were located on two barges alongside of the caisson and crib. The larger barge, 26 x 96 feet, carried two Ingersol duplex air compressors, two duplex Worthington pumps, 10 x 18, and three boilers. The other barge carried one 10¼ x 18 duplex Worthington pump, one 16 x 24, and one 18 x 30 Ingersol straight line compressor, one Knowles pump and three boilers and one dynamo with four arc lights of 1 200 candle-power each. Both barges contained machine shop requisites for such work as was found necessary to be done for the repair and maintenance of the entire plant. The air lock was located in the 4-foot cylinder of ⅜-inch boiler iron running from the top of the working chamber through the deck and crib work about 18 or 20 feet above the top of the latter. As the caisson was sunk the cylinder was carried up section by section and masonry built around it. The inside of the cylinder carried a la lder 15 inches wide with ¾-inch round rungs 16 inches apart. The upper and lower doors or valves of the air lock both swung downward. When a section was added to the cylinder, which carried the air lock, the upper door was generally made a lower. In this manner the air lock was gradually carried up as the height of masonry increased. Each door or valve was 20 x 27 inches, built of ½-inch wrought plate, to which was riveted a 1½ x ½-inch gasket ring and a 1½-inch rubber gasket, making when closed a tight fit. These doors were raised and lowered by block and tackle. As the caisson descended and air pressure increased the time required for the equalization of pressure in the air lock varied from one-half to three minutes according to the pressure and to the ability of the parties descending to sustain the corresponding changes of pressure. A concrete shaft 1 foot 6 inches diameter of ¼-inch boiler iron with a door at top and bottom was located near the middle of the caisson. The equal-

izing in this shaft was done from above by means of a 4-inch pipe connection with the working chamber to give compressed air when the concrete was passed downward. It was only used after the caisson was in place for the purpose of supplying concrete to fill the working chamber. Both the air and concrete shafts and also the 4-inch pipes were bolted to the deck course forming the roof of the working chamber. Four-inch pipes were used to discharge gravel and sand from the working chamber. Small piles of gravel and sand would be gathered at the lower openings of the pipes, which projected 6 or 7 feet into the chamber, after which a valve in the pipe would be opened and the material forced up into the pipe and into the river by the compressed air. This method was pursued nearly the whole time and removed most of the material. The larger boulders, rocks, etc., however, were hoisted through the excavating shaft located near the concrete shaft. It was carried up through the deck and crib work about 3 feet square with its lower end terminating in cylinders about 4 feet long and 2 feet 8 inches in diameter, with two doors 1 foot and 1 foot 6 inches on opposite sides. This cylinder was bolted to the top of the working chamber and was entirely within it. The bottom was concave with a small hole 3 x 4 inches just above it. The material was raised in a bucket fitting closely inside this cylinder by means of a derrick on the boat alongside the caisson. There were two openings corresponding to the doors of the cylinder on the upper part of the bucket into which the boulders, etc., were thrown, after the air in the bucket had been equalized and the doors in the cylinder opened. The bucket was guided through the shaft by two beams projecting over its top, fitting into grooves in the sides of the shaft. For a short time at the beginning of the work this excavating shaft was not used, and boulders were carried down with the caisson, except a small quantity which was locked up through the air shaft in sacks. A sand pump attached to the Worthington pump above and to the ejector below, terminating in a 4-inch rubber hose and strainer, was in frequent use to remove sand, etc., the materials having been stirred up with a jet attached to the ejector just above it. Men worked in eight-hour shifts day and night until work was finished. Valves were attached to the lower end of the 4-inch pipes leading from the compressors to the air chamber with automatic closure, in order to give the men ample time to get out if accidents should happen to the air compressor.

As already stated, the work of sinking the caissons was begun July 1st and continued without serious interruption until October 12th, when the depth of cutting edge below low water was 52 feet 9 inches, with air pressure 23 pounds per square inch. At this depth the pressure on the longitudinal walls was so great as to show some bending of the middle transverse bracing, and as bed rock was found at a depth of 1 foot 9 inches only below the cutting edge at this stage of the work, it was deemed advisable to attempt no further sinking of the caisson. The working chamber was then filled in the following manner: A solid concrete wall was built in the middle of the caisson at a weak point, with the bottom of the wall carried down to the first ledge of limestone. Excavation was then made from the entire cutting edge to bed rock and the whole carefully sealed with concrete. This was done in 10 feet sections by first carefully cleaning away all *débris* of the soapstone ledge and the thin ledges of limestone overlying the bed-rock, and putting in the concrete—the cement used being Alsen's German. After the entire cutting edge had been carefully sealed in this manner the entire working chamber was thoroughly cleaned out with great care. The soapstone ledge and the first two thin layers of limestone which overlaid the bed-rock of limestone were entirely removed. The working chamber was then completely filled with concrete, leaving no voids or interstices. The concrete shaft and air lock were then filled with concrete, using for both the working chamber and for the latter Louisville cement.

The pneumatic work of the Ohio caisson was finished on October 31st at 4 P.M., the masonry of the pier being 22½ feet high. The caisson rests on bed-rock, and its position is precisely right. It was originally placed 12 inches up-stream, with the anticipation of its being drifted that much down-stream before work was completed, and the expectation was exactly realized.

The *resumé* of the work is as follows:

Launched June 2d.
Sunk to bottom June 20th.
Began sinking caisson July 1st.
Stopped sinking caisson, October 12th.
Completed October 31st.
Time of sinking, 104 days; or 6 inches per day on the average from low water.
Total time occupied 133 days, from time caisson was sunk until completed.

Quantities in Ohio caisson and crib:

527 500 feet B. M. of timber.
3 648.6 cubic yards concrete.
155 295.5 cubic feet displacement; or 5 276.6 cubic yards.
4 651 barrels of Louisville cement.
496 barrels Alsen's German cement.
Cutting edge 52 feet 9 inches below low water.
Bed-rock 54 feet 6 inches below low water.
Bed of river to cutting edge 45 feet 9 inches.

Kentucky Caisson.

The Kentucky caisson was launched on the 10th of June in the same manner as that for the Ohio end of the 550 feet span. On July 10th it was put in position beyond the protection piles with the deck complete. On July 13th the concreting of the crib was begun, and on July 16th, with seven courses of the crib complete, containing 311 cubic yards of concrete, the displacement of the upper end was 10 feet 2 inches, and that of the lower 8 feet 0 inches. On July 21st the false bottom was entirely out and caisson nearly full of water. In this position it was found to rest on a nest of sunken logs, some of them 2 feet in diameter, running under the cutting edge. The cutting out of these logs was at once begun, and was necessarily found to be somewhat tedious and troublesome. They were cut into small pieces and taken out through the excavation shaft in buckets. This work was completed on July 28th. The presence of these logs occasioned the loss of about two weeks in sinking the caisson. In other respects the sinking of this caisson was quite similar in all its features to the other. The material passed through was sand, gravel and large boulders, being apparently through the original bed of the river after getting down some 20 feet.

The *resumé* of this portion of the work is as follows:

Launched June 10th.
Sunk June 11th.
Began sinking August 5th.
Began crib July 10th.
Completed crib September 8th.
Began concreting July 13th.
Completed concreting September 15th.
Reached bed-rock October 27th.
Completed work November 8th.
Total working days from time of location, 120.
Days employed sinking, August 5th to September 27th, 93 days.
Days employed on crib, 60; average, .59 feet per day.
Days employed on concrete, 64; average, .55 feet per day.
Average per day sinking, August 5th to September 27th, viz.:
Distance bed-rock to low water, 53.5 feet; average, 0.575 feet per day.

Distance bed-rock to bed of river, 42 feet; average, 0.451 feet per day.

CONTENTS.

514 033 feet B. M. timber.
3 569.73 cubic yards concrete.
153 383.73 cubic feet displacement.
5 155.735 cubic yards volume.
4 550 barrels Louisville cement.
450 barrels Alsen's German Portland.

The Kentucky caisson remained in its first position about 12 inches up-stream.

No delay, except two weeks cutting logs and a short delay owing to insufficient weight on top, was experienced. The work went on smoothly all the time and was a perfect success.

The heat in both caissons was at times very great, and a few men were disabled by the so-called "bends," but no lives were lost, nor were the men apparently injured from working eight hours each shift continuously.

The following is a succinct statement of the masonry built upon the cribs over the two caissons at the ends of the 550 feet span:

Began Ohio pier September 24th, 1887.
Completed Ohio pier June 30th, 1888.
Began Kentucky pier September 17th, 1887.
Completed Kentucky pier June 9th, 1888.
Actual working days, about 130 Ohio, and 140 Kentucky pier.
Rate per day, approximately, 7½ to 8 inches per day in height.

CONTENTS.

Limestone	3 431.603	cu. yds.	Kentucky pier.
Freestone	1 465.910	"	"
Oolitic stone	66.150	"	"
Total	4 963.663	cubic yards.	
Limestone	3 405.342	cu. yds.	Ohio pier.
Freestone	1 398.680	"	"
Oolitic stone	66.150	"	"
Total	4 861.172	cubic yards.	

The total weights, including lumber in substructure, concrete, masonry, iron and steel of spans, timber floor of same and maximum moving loads, on the various abutment and river piers of this structure, and the loads carried per pile on the abutment piers, and per square foot at bottom of caissons for the two river piers are as follows:

Ohio abutment pier, total weight.....	13 202 324	pounds.
Load per pile.....	77 200	"
Kentucky abutment pier, total weight..	13 890 224	"
Load per pile.....	81 200	"
Ohio River pier, total weight.....	36 719 285	"
Total load per square foot.....	13 000	"
Kentucky River pier, total weight.....	36 922 285	"
Total load per square foot.....	13 047	"

The above total weights, sustained by the two river piers, are the actual total loads, less the buoyant effect of the displacement, the volume of which is given in the preceding data.

The pneumatic portion of the substructure, including all caisson and crib work, was performed by Messrs. Sooysmith & Co., during 1887 and 1888, in their usual efficient and successful manner.

Ohio River Bridge between Covington and Cincinnati.—Specifications for the Main Spans.—By the Phœnix Bridge Company.

The bridge proper shall consist of a central span of five hundred and forty-five (545) feet, center to center of end pins; and two side spans, each four hundred and eighty-six (486) feet, center to center of end pins.

The structure shall be built for a double track railway, with a wagon-way and a sidewalk on each side.

The clear distance between trusses shall be twenty-seven (27) feet, and the clear headway, measuring from base of rail to lowest part of the overhead bracing, not less than twenty (20) feet.

The distance from center to center of railway tracks shall be thirteen (13) feet.

The width in clear for each wagon-way shall be eleven (11) feet, and that for each sidewalk five (5) feet.

Strong outside railings, each weighing not less than sixty (60) pounds per lineal foot shall be provided for the sidewalks.

Wooden screens ten (10) feet high shall be provided between the wagon-ways and the railway tracks.

All parts of the structure, excepting cross-ties, guard timbers and flooring, shall be of iron or steel, the kind of metal for each member or class of members to be used being noted on the diagram of stresses.

This diagram shall also show the maximum stresses and sectional areas of all main members of the structure.

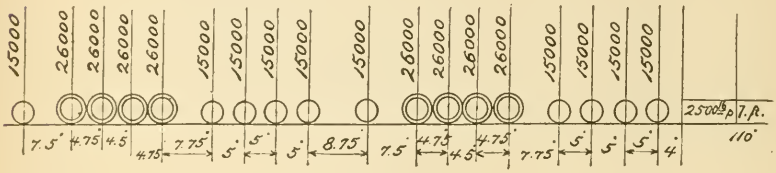
Provision shall be made for the free expansion and contraction of all parts, corresponding to a variation in temperature of one hundred and fifty (150) degrees Fahr.

As far as practicable all parts of the structure shall be accessible for inspection and painting.

Live load.

The live load shall consist—

First.—For each railway track of two consolidation engines, as per following diagram, followed by a train weighing twenty-five hundred (2 500) pounds per lineal foot.



Second.—For each wagon-way and sidewalk, of a uniform load of sixty (60) pounds per square foot, which is equivalent to nine hundred and sixty (960) pounds per lineal foot on each truss, both wagon-ways and sidewalks being considered fully loaded when calculating the stresses in the main members of trusses.

Third.—In calculating the stresses in the floor-system and primary truss members, in addition to the railway load above specified, the following loading is to be applied alternately on each side of the bridge:

Fifteen (15) tons uniformly distributed on ten (10) lineal feet of wagon-way, followed and preceded by a uniform load of eighty (80) pounds per square foot; and a uniform load of eighty (80) pounds per square foot on the sidewalks.

To provide for the effect of impact on all parts of the structure liable to sudden loading and vibration, additions to the stresses produced by the railway live load shall be made as follows: For floor beam hangers and riveted connections of floor beams and stringers, fifty (50) per cent.; for stringers, floor beams, suspenders, primary struts, counters and posts strained mainly by counters, twenty-five (25) per cent.

Dead load.

In determining the total weight of the structure for the purpose of calculating stresses, the weight of the iron shall be assumed at the rate of three and a third ($3\frac{1}{3}$) pounds per lineal foot of a bar of one (1) square inch area; and the weight of the steel to be assumed to be two (2) per cent. greater than that of the iron.

Creosoted timber shall be assumed to weigh five (5) pounds per foot, board measure; oak and yellow pine, four (4) pounds per foot, board measure; and white pine three (3) pounds per foot, board measure.

An addition of fifty (50) pounds per lineal foot for each track shall be made for the weight of rails, spikes and joints.

The assumed and actual dead loads shall agree within three (3) per cent.

Wind pressure shall be calculated for thirty (30) pounds per square foot on the exposed surfaces of both trusses and on the vertical projection of the floor system (the wooden screens being omitted), and on a moving train surface averaging 10 square feet per lineal foot of bridge. Wind pressure.

The co-efficient of sliding friction of shoe plates upon rollers or bed plates shall be assumed to be twenty-five (25) per cent. Friction.

All parts of the structure shall be so proportioned that the combined effects of temperature and all the specified loads, except the wind pressure, shall not cause the stress per square inch to exceed the following limits: Limits of intensities of working stresses.

In tension, ten thousand (10 000) pounds. For iron.

In compression, for lengths less than fifty (50) times the least radius of gyration, eight thousand (8 000) pounds.

In shearing across fibers, seven thousand five hundred (7 500) pounds.

In bending in the extreme fibers of pins, fifteen thousand (15 000) pounds.

On bearing surfaces, twelve thousand (12 000) pounds.

The bearing surfaces of pins and rivets shall be reckoned from the diameter and not from the semicircle.

The stress per square inch in compression for members whose lengths exceed fifty (50) times their least radii of gyration shall be reduced according to the following formulas:

For square bearings..... $R = \dfrac{8\,000}{1 + \dfrac{1}{36\,000}\,\dfrac{l^2}{r^2}}$

For pin bearings..... $R = \dfrac{8\,000}{1 + \dfrac{1}{18\,000}\,\dfrac{l^2}{r^2}}$

For the flanges of rolled beams..... $R = \dfrac{10\,000}{1 + \dfrac{1}{5\,000}\,\dfrac{l^2}{b^2}}$

For the top flanges of built beams... $R = \dfrac{8\,000}{1 + \dfrac{1}{5\,000}\,\dfrac{l^2}{b^2}}$

where R is the intensity of working stress, l length in inches of members between supports, r least radius of gyration of cross-section, b breadth of top flange of girder in inches.

In tension on chord bars and end main diagonals, sixteen thousand (16 000) pounds. For steel.

In shearing on rivets and pins, ten thousand (10 000) pounds.

In bending on the extreme fibers of pins, twenty thousand (20 000) pounds.

On bearing surfaces, fifteen thousand (15 000) pounds for rivets and eighteen thousand (18 000) pounds for pins.

On main diagonals nearest the middle of the span, thirteen thousand (13 000) pounds tension.

For intermediate main diagonals the tensile intensities are to be directly interpolated.

In compression on top chords and inclined end posts, provided that the ratio of length to least radius of gyration does not exceed fifty (50), fourteen thousand (14 000) pounds.

For all other steel struts the intensities are to be found by the following formula:

$$R = \frac{14\,000}{1 + \frac{1}{20\,000}\,\frac{l^2}{r^2}}$$

where R is the intensity, l the length of column in inches, and r the least radius of gyration in inches.

Alternating stresses.

Steel struts subject to alternating stresses of compression and tension shall be proportioned by the following formula:

$$R = \frac{14\,000\left(1 - \frac{1 \text{ min. stress}}{2 \text{ max. stress}}\right)}{1 + \frac{1}{20\,000}\,\frac{l^2}{r^2}}$$

where R, l and r have the same signification as in the last clause.

Wind stresses.

An addition of fifty (50) per cent. to all specified intensities of working stresses shall be allowed for all wind stresses and for all combinations of wind stresses with other stresses.

Compression members.

The thickness of metal in compression shall not be less than one-sixteenth ($\frac{1}{16}$) of the distance between supports in line of stress, or less than one-thirtieth ($\frac{1}{30}$) of the distance between supports at right angles to the line of stress, or less than one-eighth ($\frac{1}{8}$) of the distance from the edge of plate of flange to line of support, or less than one-quarter ($\frac{1}{4}$) inch when both faces are accessible for painting, or less than five-sixteenths ($\frac{5}{16}$) of an inch when only one face is accessible for painting.

The ratio of length of strut between supporting points to its least diameter shall not exceed forty-five (45).

Eye-bars and upset rods.

The eyes and threaded parts of bars and rods shall be so proportioned as to develop the full strength of the members. The heads shall not be welded to the body of the bar.

Long tension members shall be supported at suitable intervals to avoid rattling and undue stress by bending. No lateral or sway-rod shall be less than one (1) inch in diameter.

Pins and rivets.

Pins shall be proportioned to resist the bending as well as the shearing forces acting upon them.

The limits of stress specified for shearing and for the pressure on

bearing surface of holes shall determine the number and size of the rivets. Plate girders.

In designing plate girders the office of the web shall be to resist shear only, and that of the flanges to resist bending only.

The thickness of the web shall never be less than a quarter ($\frac{1}{4}$) of an inch, and it shall be in all cases properly stiffened.

Connections and attachments of all members shall be so designed that the stress on each member can be correctly calculated. Whenever practicable, the lines of stress shall coincide with the lines of centers of gravity of cross-sections, and shall intersect at the joint point; and in cases where these conditions cannot be complied with, provision shall be made for a stiff connection between the members thus meeting. Connections and attachments.

Bed plates and bearing plates shall be truly planed on all sliding and rolling surfaces, and shall be so proportioned that the maximum pressure per square foot on the masonry will not exceed thirty-six thousand (36 000) pounds. They shall be securely anchored against upward and sideway motion. Bed plates and friction rollers.

The rollers shall be of steel; the pressure per lineal inch on same shall not exceed $\sqrt{540\ 000\ d}$, where d is the diameter of the rollers in inches.

The rollers and the rolling surface of bed plates shall be protected by wrought-iron casing to keep out foreign matters. Adjacent ends of consecutive spans shall have a common bed plate of wrought-iron at each side of bridge.

One end of each truss is to be firmly anchored to the masonry, and the other end so anchored as to prevent upward and sideway motion, but permitting longitudinal motion. Anchorage.

Quality of Materials.—Wrought-iron.

All wrought-iron must be tough, ductile, fibrous, and of a uniform quality for each class, straight, smooth, free from cinder pockets or injurious flaws, buckles, blisters or cracks. Character and finish.

As the thickness of the bar approaches the maximum that the rolls will produce, the same perfection of finish will not be required as in the thinner ones. No specific process or provision of manufacture will be demanded, provided the material fulfills the requirements of this specification.

3. The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece, not less than $\frac{1}{4}$ inch in thickness, cut from the full size bar, and planed and turned parallel; if the cross-section is reduced, the tangent between shoulders shall be at least twelve times its shortest dimension, and the area of the minimum cross-section in either case shall not be less than $\frac{1}{4}$ of a square inch and not more than 1 square inch. Whenever practicable, two Standard test pieces.

opposite sides of the piece are to be left as they come from the rolls, but the finish of opposite sides must be the same in this respect. A full-size bar, when not exceeding the above limitations, may be used as its own test piece. In determining the ductility the elongation shall be measured, after breaking, on an original length the nearest multiple of a $\frac{1}{4}$ inch to ten times the shortest dimension of the test piece, in which length must occur the curve of reduction from stretch on both sides of the point of fracture, but in no case on a shorter length than 5 inches.

Tension iron for open trusses.

4. All iron to be used in the tensile members of open trusses, laterals, pins and bolts, except plate iron over 8 inches wide and shaped iron, must show by the standard test pieces a tensile strength in pounds per square inch of—

$$52\,000 \times \frac{7\,000 \text{ area of original bar}}{\text{circumference of original bar}} \text{ (all in inches),}$$

with an elastic limit not less than one-half the strength given by this formula, and an elongation of 20 per cent.

Plate iron.

Plate iron 24 inches wide and under, and more than 8 inches wide, must show by the standard test pieces a tensile strength of 48 000 pounds per square inch, with an elastic limit not less than 26 000 pounds per square inch, and an elongation of not less than 12 per cent. All plates over 24 inches in width must have a tensile strength not less than 46 000 pounds per square inch, with an elastic limit not less than 26 000 pounds per square inch.

Plates from 24 to 36 inches in width must have an elongation of not less than 10 per cent.; and those from 36 to 48 inches in width, 8 per cent.; over 48 inches in width, 5 per cent.

Shaped iron.

All shaped iron and other iron not hereinbefore specified must show by the standard test pieces a tensile strength in pounds per square inch of—

$$50\,000 \times \frac{7\,000 \text{ area of original bar}}{\text{circumference of original bar,}}$$

with an elastic limit of not less than one-half the strength given by this formula, and an elongation of 15 per cent. for bars $\frac{5}{8}$ of an inch and less in thickness, and of 12 per cent. for bars of greater thickness.

Hot bending.

All plates, angles, etc., which are to be bent hot, in the manufacture, must, in addition to the above requirements, be capable of bending sharply to a right angle at a working heat without sign of fracture.

Rivet iron.

All rivet iron must be tough and soft, and pieces of the full diameter of the rivet must be capable of bending cold until the sides are in close contact without sign of fracture on the convex side of the curve.

All iron specified in clause 4 must bend cold, 180 degrees without sign of fracture, to a curve the inner radius of which equals the thickness of the piece tested.

Specimens of full thickness cut from plate iron, or from the flanges or webs of shaped iron, must stand bending cold through 90 degrees, to a curve the inner radius of which is one and a half times its thickness, without sign of fracture.

11. For each contract four standard test pieces and one additional for each 50 000 pounds of wrought-iron will, if required, be furnished and tested by the contractor without charge, and if any additional test pieces are required by the purchaser, they will be made for him at the rate of $5 each; or, if the contractor desires additional tests, they shall be made at his own expense under the supervision of the purchaser, the quality of the material to be determined by the result of all the tests in the manner set forth in the following clause: Number of test pieces.

The respective requirements stated are for an average of the tests for each, and the lot of bars or plates from which samples were selected shall be accepted if the tests give such average results, but if any test piece gives results more than 4 per cent. below said requirements, the particular bar from which it was taken may be rejected, but such tests shall be included in making the average. If any test piece has a manifest flaw its test shall not be considered. For each bar thus giving results more than 4 per cent. below the requirements tests from two additional bars shall be furnished by the contractor without charge, and if in a total of not more than ten tests two bars (or, for a larger number of tests, a proportionately greater number of bars) show results more than 4 per cent. below the requirements, it shall be cause for rejecting the lot from which the sample bars were taken. Such lots shall not exceed 20 tons in weight, and bars of a single pattern, plates rolled in universal mill or in grooves, and sheared plates, shall each constitute a separate lot.

The inspection and tests of the material will be made promptly on its being rolled, and the quality determined before it leaves the rolling mill. All necessary facilities for this purpose shall be afforded by the manufacturer; but, if the inspector is not present to make the necessary tests, after due notice given him, then the contractor shall proceed to make such number of tests on the iron then being rolled as may have been agreed upon; or, in the absence of any special agreement, the number provided for in clause 11, and the quality of such material shall be determined thereby. Time of inspection.

A variation in cross-section or weight of rolled material of more than 2½ per cent. from that specified may be cause for rejection. Variation of Weight.

Steel.

No specific process or provision of manufacture will be demanded, provided the material fulfills the requirements of this specification. The ultimate tensile resistance of the steel to be used in tension shall be Test bars.

62 500 pounds per square inch, and the ultimate tensile resistance of the steel to be used in compression shall be 68 000 pounds per square inch. The tests to be made in the following manner:

16. From one ingot of each cast a round sample bar, not less than ¾ of an inch in diameter, and having a length not less than twelve diameters between jaws of testing machine, shall be furnished and tested by the manufacturer without charge. These bars are to be truly round, and shall be finished at a uniform heat, and arranged to cool uniformly, and from these test pieces alone the quality of the material shall be determined, as follows:

Tensile tests.

17. All the above-described test bars must have a tensile strength within 4 000 pounds per square inch of that specified; an elastic limit not less than one-half of the tensile strength of the test bar; a percentage of elongation not less than 1 200 000 ÷ the tensile strength in pounds per square inch, and a percentage of reduction of area not less than 2 400 000 ÷ the tensile strength in pounds per square inch. In determining the ductility, the elongation shall be measured after breaking on an original length of ten times the shortest dimension of the test piece, in which length must occur the curve of reduction from stretch on both sides of the point of fracture.

Finish and reduction of area on finished bars.

Finished bars must be free from flaws or cracks and must have a workmanlike finish, and round or square test pieces cut therefrom when pulled asunder shall have a reduction of area at the point of fracture as above specified.

Number of test pieces.

19. For each contract four such tests for reduction of area and four for bending, and one additional of each for each 50 000 pounds of steel, will, if required, be made by the contractor without charge, and if the purchaser is not satisfied that the reduction of area test correctly indicates the effect of the heating and rolling, such additional tests for tensile strength, limit of elasticity and ductility as he may desire will be made for him on test pieces conforming to the provisions of clause 3 at the rate of $5 each, or, if the contractor desires additional tests, he may make them at his own expense, under the supervision of the purchaser, the quality of the material to be determined by the result of all the tests in the manner set forth in the following clause:

Except for tensile strength, the respective requirements stated are for an average of the tests for each, and the lot of bars or plates from which samples were selected shall be accepted if the tests give such average results, but, if any test piece gives results more than 4 per cent. below said requirements, the particular bar from which it was taken may be rejected, but such tests shall be included in making the average.

If any test piece has a manifest flaw, its test shall not be considered. For each bar thus giving results more than 4 per cent. below the requirements tests from two additional bars shall be furnished by the contractor without charge, and if in a total of not more than ten tests two bars

(or, for a larger number of tests, a proportionately greater number of bars) show results more than 4 per cent. below the requirements, it shall be cause for rejecting the lot from which the sample bars were taken. Such lots shall not exceed 20 tons in weight, and bars of a single pattern, plates rolled in universal mill or grooves, and sheared plates shall each constitute a separate lot.

Rivet steel shall have a specified tensile strength of 60 000 pounds per square inch, and test bars must have a tensile strength within 4 000 pounds per square inch of that specified, and an elastic limit, elongation and reduction of area at the point of fracture, as stated in clause 17, and be capable of bending double, flat, without sign of fracture on the convex surface of the bend. Rivet steel.

The inspection and tests of the material will be made promptly on its being rolled, and the quality determined before it leaves the rolling-mill. All necessary facilities for this purpose shall be afforded by the manufacturer; but, if the inspector is not present to make the necessary tests, after due notice given him, then the contractor shall proceed to make such number of tests on the steel then being rolled as may have been agreed upon; or, in the absence of any special agreement, the number provided for in clauses 16 and 19, and the quality of such material shall be determined thereby. Time of inspection.

A variation in cross-section or weight of rolled materials of more than 2½ per cent. from that specified may be cause for rejection. Variation of weights.

Cast-iron.

Except where chilled iron is specified, all castings shall be tough gray iron, free from injurious cold shuts or blow-holes, true to pattern and of a workmanlike finish. Sample pieces 1 inch square cast from the same heat of metal in sand moulds shall be capable of sustaining on a clear span of 4 feet 6 inches a central load of 500 pounds when tested in the rough bar.

Workmanship.

Inspection of the work shall be made as it progresses, and at as early a period as the nature of the work permits. Inspection.

All workmanship must be first-class. All abutting surfaces of compression members, except flanges of plate girders where the joints are fully spliced, must be planed or turned to even bearings so that they shall be in such contact throughout as may be obtained by such means. All finished surfaces must be protected by white lead and tallow.

The rivet holes for splice plates of abutting members shall be so accurately spaced that when the members are brought into position the holes shall be truly opposite before the rivets are driven.

When members are connected by bolts which transmit shearing

strains the holes must be reamed parallel, and the bolts turned to a driving fit.

Rollers must be finished perfectly round and roller-beds planed.

Rivets.

Rivets must completely fill the holes, have full heads concentric with the rivet, of a height not less than 0.6 the diameter of the rivet, and in full contact with the surface, or be countersunk when so required, and machine driven wherever practicable.

Eye-bars and pin-holes.

Built members must, when finished, be true and free from twists, kinks, buckles or open joints between the component pieces.

All pin holes must be accurately bored at right angles to the axis of the members, unless otherwise shown in drawings, and in pieces not adjustable for length no variation more than $\frac{1}{32}$ of an inch will be allowed in the length between centers of pin holes. The diameter of the pin holes shall not exceed that of the pins by more than $\frac{1}{32}$ inch, nor by more than $\frac{1}{50}$ inch for pins under $3\frac{1}{2}$ inches diameter. Eye-bars must be straight before boring; the holes must be in the center of the heads, and on the center line of the bars.

Whenever links are to be packed more than $\frac{1}{8}$ of an inch to the foot of their length out of parallel with the axis of the structure, they must be bent with a gentle curve until the head stands at right angles to the pin in their intended position before being bored. All links belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing. No welds will be allowed in the body of the bar of eye-bars, laterals or counters, except to form the loops of laterals, counters, and sway-rods. Eyes of laterals, stirrups, sway-rods and counters must be bored. Pins and lateral bolts must be finished perfectly round and straight, and the party contracting to erect the work must provide pilot nuts where necessary to preserve the threads while the pins are being driven. Thimbles or washers must be used whenever required to fill the vacant spaces on pins or bolts.

Pilot nuts.

Test of eyes on full-size bars.

To determine the strength of eyes, full-size eye-bars or rods with eyes may be tested to destruction, provided notice is given in advance of the number and size required for the purpose, so that the material can be rolled at the same time as that required for the structure, and any lot of iron bars from which full-size samples are tested shall be accepted.

First.—If not more than one-third of the bars tested break in the eye; or,

Second.—If more than one-third do break in the eye and the average of the tests of those which so break shows a tensile strength in pounds per square inch of original bar, given by the formula:

$$52\,000 = \frac{7\,000 \text{ area of original bar}}{\text{Circumference of original bar}} = 500 \text{ width of bar (all in inches)},$$

and not more than one-half of those which break in the eye fail at more than 5 per cent. below strength given by the formula. Any lot of steel bars from which full-size samples are tested shall be accepted if the average of the tests shows a strength per square inch of original bar, in those which do break in the eye, within 4 000 pounds of that specified in clause 17; but if one-half the full-size samples break in the eye, it shall be cause for rejecting the lot from which the sample bars were taken.

All full-size sample bars which break in the eye at less than the strength here specified shall be at the expense of the contractor, unless he shall have made objection in writing to the form or dimension of the heads before making the eye-bars. All others shall be at the expense of the purchaser. If the contractor desires additional tests they shall be made at his own expense, under the supervision of the purchaser, the acceptance of the bars to be determined by the result of all the tests in the manner above set forth. A variation from the specified dimensions of the heads will be allowed, in thickness of $\frac{1}{32}$ inch below and $\frac{1}{16}$ above that specified, and in diameter of $\frac{1}{4}$ inch in either direction; but if the dimensions of any head fall below those here allowed, such bar may be tested in accordance with the provisions of this section to determine whether, if rejected, it shall be at the expense of the purchaser or of the contractor.

Rivet holes must be accurately spaced; the use of drift pins will be allowed only for bringing together the several parts forming a member, and they must not be driven with such force as to distort the metal about the holes; if the hole must be enlarged to admit the rivet, it must be reamed; all rivet holes in steel work, if punched, shall be made with a punch $\frac{1}{8}$ inch in diameter less than the diameter of the rivet intended to be used, and shall be reamed to a diameter $\frac{1}{16}$ inch greater than the rivet. Punching and reaming.

In all cases where a steel piece in which the full length is required has been partially heated, the whole piece must be subsequently annealed. Annealing.

All bends in steel must be made cold, or if the degree of curvature is so great as to require heating, the whole piece must subsequently be annealed.

All surfaces inaccessible after assembling must be well painted or oiled before the parts are assembled. The decision of the engineer shall control as to the interpretation of drawings and specifications during the execution of work thereunder, but this shall not deprive the contractor of his right to redress, after the completion of the work, for an improper decision. Painting.

In the effective area of riveted members, pin, bolt and rivet holes shall be counted out for tension, and bolt and pin holes shall be counted out for compression. Rivets are not to be used in direct tension. Riveted work.

Rivets shall be used in preference to bolts for all rigid connections to resist shearing.

The webs of plate girders shall be spliced with a plate on each side.

The space between the edge of the piece and the edges of rivet holes shall be such that the metal will not crack or split by punching and riveting.

Adjustment.

Lateral and sway-rods and all other members requiring adjustment shall be provided with adjusting screw threads and check nuts, convenient of access.

Sway bracing.

The whole structure shall be thoroughly sway-braced, both horizontally and vertically.

Bolts, washers and nuts.

Washers and nuts shall have a uniform bearing. All nuts shall be easily accessible with a wrench for the purpose of adjustment. No round-headed bolts will be allowed. All bolts through wood must be provided with wrought-iron washers under heads and nuts.

Railway floor.

The cross-ties shall be eight (8) inches by eight (8) inches by twenty-two (22) feet, spaced fourteen (14) inches from center to center.

They shall be fastened by five-eighths ($\frac{5}{8}$) inch lag screws to guard timbers six (6) inches by eight (8) inches in section.

These guard timbers shall be lap-jointed and placed so that their central vertical planes shall be thirteen (13) inches outside of the central vertical planes of the outer rails.

Every fourth tie shall be bolted to the stringers by three-quarter ($\frac{3}{4}$) inch bolts.

Roadways.

The ties and guard timbers shall be practically uniform in thickness.

The stringers shall be spaced about three (3) feet four (4) inches from center to center.

The flooring of the wagon-ways shall consist of two thicknesses of plank.

The under flooring shall be of creosoted timber planks three (3) inches thick, fastened on nailing pieces, also creosoted, secured on the top of the stringers.

The top flooring shall consist of white oak or yellow pine two and one-half ($2\frac{1}{2}$) inches thick and not more than eight (8) inches wide.

The flooring of the sidewalks shall be of white oak or yellow pine two (2) inches thick and not more than eight (8) inches wide, fastened to nailing pieces secured on top of stringers.

The under flooring of the wagon-ways and the sidewalk flooring shall be of the uniform thickness specified.

The under flooring of the wagon-way shall be fastened to the nailing pieces with seven (7) inch by seven sixteenth ($\frac{7}{16}$) inch wrought spikes.

The top flooring of the wagon-way shall be fastened by fifty (50) penny and the sidewalks by forty (40) penny nails of the best quality.

The stringers shall be securely fastened to the floor beams and in such a manner as to secure the requisite stiffness of the floor system.

The stringers for the railway floor shall be placed between the floor beams so as to reduce the height from base of rail to bottom of floor beams to a minimum.

Two wheel guards eight (8) inches by ten (10) inches, of white oak, shall be provided on each wagon-way. They shall be lap-jointed and securely bolted to the flooring by three-quarter ($\frac{3}{4}$) inch bolts placed four (4) feet apart.

Camber.

The camber measured on the center line of pins of chords shall not be less than one eighteen hundredth ($\frac{1}{1800}$) of the span. The track ties shall be notched over the stringers sufficiently to secure them firmly in place.

Screens.

The screens between the wagon-ways and railway shall be of tongued and grooved pine plank not more than four (4) inches wide, strongly built, and supported so as to resist a wind pressure of thirty (30) pounds per square foot. They shall be painted on both sides with three coats of approved metallic paint.

Railings.

The outside railings shall be of approved design, and not less than four (4) feet high above floor of sidewalks.

They shall be supported directly by the floor-beam brackets and braced laterally with strong outside stays riveted thereto. Intermediate stays not more than ten (10) feet apart shall also be provided. The railings and floor timbers shall extend over all piers and make suitable connections with the masonry at the ends of the structure.

Timber.

The timber shall be of first-class sound wood of the kinds specified. It shall be sawed true and out of wind, full-sized, free from wind shakes, large or loose knots, decayed brash or sap-wood, worm holes, or any defect impairing its strength or durability. The creosoted timber shall be prepared:

1st, by a thorough seasoning of the wood at a temperature not to exceed two hundred and thirty (230) degrees Fahrenheit, in a partial vacuum of twenty-four (24) inches of mercury, a sufficient length of time being used in this operation to avoid the cracking or splitting of the timber.

2d, by the injection into the wood under a pressure of not less than one hundred and fifty (150) pounds per square inch of not less than twelve (12) pounds of heavy creosote oil to each cubic foot of timber. This oil is to be of the best quality.

Sap-wood in timber to be creosoted will be accepted if otherwise sound.

All framing and trimming must be done before treatment.

Inspections and tests.

Competent inspectors shall inspect the material at the mills and shops and make the required tests. These inspectors shall be mutually agreed upon. Before the final estimate is paid a thorough test of the

finished structure shall be made by loading each span with the nearest equivalent load obtainable to that described under the head of loads; each span shall not deflect under such a load more than one eighteen hundredth ($\frac{1}{1800}$) of its length, and shall return to its original camber when the load is removed.

Painting.

All iron and steel before leaving the shop shall be kept as clean as practicable and shall be thoroughly coated with boiled linseed oil; and all planed or turned surfaces shall be coated with white lead mixed with tallow. After erection the entire structure, excluding timber, shall be painted with two coats of red lead, and the wooden screens shall be painted with three coats of metallic paint.

No painting shall be done in wet or freezing weather.

Staging and river navigation.

All staging and other temporary structures used in the construction of the bridge shall be removed by the contractor before payment of the final estimate. River navigation and circulation on streets and railway tracks in the neighborhood of the bridge site shall be maintained during erection, the contractor complying in such matters with any regulation or condition that may be imposed by the United States Engineer in charge of the Ohio River, by the city authorities, or by the railway companies.

Risks.

The contractor shall assume all risks from floods, storms, damage to persons and property, and casualties of every description, and shall furnish all materials, labor and machinery necessary or incidental to or in any way connected with the manufacture, transportation, erection and maintenance of the structure until its final acceptance, without additional compensation.

Supplementary Specifications for Steel Eye-bars to be Furnished by the Phœnix Bridge Company.

3. The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece, not less than ¼ inch in thickness, cut from the full-sized bar, and planed or turned parallel; if the cross-section is reduced, the tangent between shoulders shall be at least twelve times its shortest dimension, and the area of minimum cross-section in either case shall not be less than ¼ of a square inch and not more than 1 square inch. Whenever practicable, two opposite sides of the piece are to be left as they come from the rolls, but the finish of opposite sides must be the same in this respect. A full-sized bar, when not exceeding the above limitations, may be used as its own test piece. In determining the ductility the elongation shall be measured, after breaking on an original length the nearest multiple of a ¼ inch to ten times the shortest dimension of the test piece, in which length must occur the curve of reduction from stretch on both sides of the point of fracture, but in no case on a shorter length than 5 inches.

4. The ultimate tensile resistance of the steel to be used in tension shall be 62 500 pounds per square inch.

16. From one ingot of each cast a round sample bar not less than ¾ of an inch in diameter, and having a length of not less than twelve diameters between jaws of testing machine, and two pieces cut from the finished bars for each cast and prepared as in clause 3 shall be furnished and tested by the manufacturer without charge. The bars from the ingot are to be truly round, and shall be finished at a uniform heat, and arranged to cool uniformly.

The tests from the finished bars shall be made as set forth in clause 3. From these tests the quality of the material shall be determined as follows:

17. All the above-described test bars must have a tensile strength within 4 000 pounds per square inch of that specified in clause 4; an elastic limit of not less than one-half of the tensile strength of the test bar; a percentage of elongation of not less than 1 200 000 : the tensile strength in pounds per square inch, and a percentage of reduction of area not less than 2 400 000 : the tensile strength in pounds per square inch. In determining the ductility the elongation shall be measured after breaking on an original length of ten times the shortest dimension of the test piece, in which length must occur the curve of reduction from stretch on both sides of the point of fracture.

18. Finished bars must be free from injurious flaws or cracks and must have a workmanlike finish, and round or square test pieces cut therefrom when pulled asunder shall have a reduction of area at the point of fracture as above specified.

19. For each contract four such tests respectively for reduction of area and for bending, and one additional of each for each 50 000 pounds of steel, will, if required, be made by the manufacturer without charge; and if the purchaser is not satisfied that the reduction of area test correctly indicates the effect of the rolling and heating, such additional tests for tensile strength, limit of elasticity and ductility as he may desire will be made for him on test pieces conforming to the provisions of clause 3, at the rate of $5 each; or, if the manufacturer desires additional tests, he may make them at his own expense, under the supervision of the purchaser, the quality of the material being determined by the result of all the tests in the manner set forth in the following clause:

20. Except for tensile strength, the respective requirements stated are for an average of the tests for each, and the lot of bars or plates from which samples were selected shall be accepted if the tests give such average results; but, if any test piece gives results more than 4 per cent. below said requirements, the particular bar from which it was taken may be rejected, but such tests shall be included in making the average. If any test piece has a manifest flaw, its test shall not be con-

sidered. For each bar thus giving results more than 4 per cent. below the requirements, tests from two additional bars shall be furnished by the manufacturers without charge, and if in a total of not more than ten tests two bars (or, for a larger number of tests, a proportionately greater number of bars) show results more than 4 per cent. below the requirements, it shall be cause for rejecting the lot from which the sample bars were taken. Such lots shall not exceed 20 tons in weight, and bars of a single pattern shall each constitute a separate lot.

22. The inspection and tests of the material will be promptly made on its being rolled, and the quality determined before it leaves the rolling mill. All necessary facilities for this purpose shall be afforded by the manufacturer; but, if the inspector is not present to make the necessary tests, after due notice given him, then the manufacturer shall proceed to make such number of tests on the steel then being rolled as may have been agreed upon; or, in the absence of any special agreement, the number provided for in clauses 16 or 19, and the quality of such material shall be determined thereby.

23. A variation in cross-section or weight of rolled material of more than $2\frac{1}{2}$ per cent. from that specified may be cause for rejection.

32. All pin holes must be accurately bored at right angles to the axis of the members, unless otherwise shown in the drawings, and in pieces not adjustable for length no variation of more than $\frac{1}{32}$ of an inch will be allowed in the length between centers of pin holes; the diameter of the pin holes shall not exceed that of the pins by more than $\frac{1}{32}$ inch, nor by more than $\frac{1}{50}$ inch for pins under $3\frac{1}{2}$ inches diameter. Eye-bars must be straight before boring; the holes must be in the center of the heads, and on the center line of the bars.

All eye-bars belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing. No welds whatever will be allowed in any part of any eye-bar.

33. To determine the strength of the eyes, full-size bars with eyes may be tested to destruction, provided notice is given in advance to the number and size for this purpose, so that the material can be rolled at the same time as that required for the structure.

35. In all cases where a steel piece in which the full strength is required has been partially heated, the whole piece must be subsequently annealed.

For the purpose of making tests of full-sized bars cover the process of annealing as well as mode of manufacture and dimensions and shape of head; all bars of any lot of eye-bars annealed at the same time shall be designated by a special mark, and shall be considered as one "lot." A complete record of the number and size of bars in each lot shall be preserved. A lot shall also consist of all bars of whatever length, of the same dimensions of cross-section or of same dimensions of head.

Any lot of steel bars from which full-sized samples are tested shall be accepted if the average of the tests shows a strength per square inch of original bar in those which break in the eye equal to that specified in clause 17; but if one-half of the full-sized test bars belonging to any lot break in the head, all bars of such a lot may be rejected.

All bars belonging to any lot may be rejected if the test bars of that lot break in the body and show an average ultimate resistance under 58 500 pounds per square inch in original section.

All full-sized bars which break in the eye at less than the strength here specified shall be at the expense of the manufacturer, unless he shall have made objection in writing to the form or dimension of the heads before making the eye-bars.

The costs of the bars and the tests of bars which break in the head, or which break in the body of the bar at less than 58 500 pounds per square inch, or which belong to rejected lots, shall be borne by the manufacturer; the expense of other full-sized tests shall also be borne by the manufacturer, but the bars shall be paid for at contract price, less its scrap value.

Supplementary Specifications for Steel Pins for Ohio River Bridge.

The ultimate tensile resistance of the steel to be used in all pins shall be 62 500 pounds per square inch.

(3.) The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece, not less than ¼ inch in thickness, cut from the full-sized bar, and planed and turned parallel; if the cross-section is reduced, the tangent between shoulders shall be at least twelve times its shortest dimension, and the area of the minimum cross-section in either case shall not be less than ¼ of a square inch, and not more than 1 square inch. Whenever practicable, two opposite sides of the piece are to be left as they come from the rolls, but the finish of opposite sides must be the same in this respect. A full-sized bar, when not exceeding the above limitations, may be used as its own test piece. In determining the ductility the elongation shall be measured, after breaking, on an original length the nearest multiple of ¼ inch to ten times the shortest dimension of the test piece, in which length must occur the curve from reduction from stretch on both sides of the point of fracture, but in no case on a shorter length than 5 inches. Standard test pieces.

(16.) From one among the ingots of each cast two round sample bars, not less than ¾ of an inch in diameter, and having a length of not less than twelve diameters between jaws of testing machine, shall be furnished and tested by the manufacturer without charge. These bars are to be truly round, and shall be finished at a uniform heat, and arranged to cool uniformly, and from these test pieces alone the quality of the material shall be determined as follows: Test bars.

(17.) All the above-described test bars must have a tensile strength within 4 000 pounds per square inch of that specified, an elastic limit not less than one-half of the tensile strength of the test bar, a percentage of elongation not less than 1 200 000 ÷ the tensile strength in pounds per square inch, and a percentage of reduction of area not less than 2 400 000 ÷ the tensile strength in pounds per square inch. In determining the ductility the elongation shall be measured after breaking on an original length of ten times the shortest dimension of the test piece, in which length must occur the curve of reduction from stretch on both sides of the point of fracture.

Finished pins must be free from injurious flaws or cracks and must have a workmanlike finish, and round or square pieces cut therefrom when pulled asunder shall have a tensile resistance and reduction of area at the point of fracture as above specified.

(19.) For each contract four tests for tensile resistance and reduction of area and four for bending, and one additional for each 50 000 pounds of steel will, if required, be made by the contractor, without charge; and if the purchaser is not satisfied that the reduction of area test correctly indicates the effect of the heating and rolling, such additional tests for tensile strength, limit of elasticity and ductility as he may desire will be made for him on test pieces conforming to the provisions of clause 3 at the rate of $5 each; or, if the contractor desires additional tests, he may make them at his own expense, under the supervision of the purchaser, the quality of the material to be determined by the result of all the tests in the manner set forth in the following clause:

If any test piece has a manifest flaw, its test shall not be considered. For each bar thus giving results more than 4 per cent. below the requirements tests from two additional bars shall be furnished by the contractor without charge, and if in a total of not more than ten tests two bars (or, for a larger number of tests, a proportionately greater number of pins) show results more than 4 per cent. below the requirements, it shall be cause for rejecting the lot from which the sample bars were taken. Such lots shall not exceed 20 tons in weight.

The inspection and tests of the material will be made promptly on its being rolled, and the quality determined before it leaves the rolling mill. All necessary facilities for this purpose shall be afforded by the manufacturer; but, if the inspector is not present to make the necessary tests, after due notice given him, then the contractor shall proceed to make such number of tests on the steel then being rolled as may have been agreed upon; or, in the absence of any special agreement, the number provided for in clauses 16 and 19, and the quality of such material shall be determined thereby.

Rounds from which pins are to be turned must be true and straight, so that in turning to finished size the metal removed shall at no point be more then $\frac{1}{8}$ inch thick, nor at any place shall flaws or cracks be revealed by the cutting tool.

All forged pins shall be reduced from a single bloom or ingot until perfect homogeneity is secured throughout the whole mass. The finished forgings shall be free from all flaws or cracks. No forging shall be done below a red heat, and after the forging is complete each rough pin shall be thoroughly annealed.

All forged pins in the rough must be so nearly true that in turning the thickness off metal taken off must not exceed ¼ inch at any point.

Whether pins are forged or rolled, if any cracks or flaws are revealed while in the lathe by the cutting tool, such pin or pins will be rejected.

Specifications for Steel Rivets for the Cincinnati and Covington Bridge.

Rivet steel shall have a specified tensile strength of 60 000 pounds per square inch, and test bars must have a tensile strength within 4 000 pounds per square inch of that specified, and an elastic limit, elongation and reduction of area at the point of fracture, as stated below, and be capable of bending double, flat, without sign of fracture on the convex surface of the bend.

16. From one ingot of each cast two round sample bars, not less than ¾ of an inch in diameter, and having a length not less than twelve diameters between jaws of testing machine, shall be furnished and tested by the manufacturer without charge. These bars are to be truly round, and shall be finished at a uniform heat, and arranged to cool uniformly, and from these test pieces alone the quality of the material shall be determined as follows: Test bars.

17. All the above-described test bars must have a tensile strength within 4 000 pounds per square inch of that specified, an elastic limit not less than one-half of the tensile strength of the test bar, a percentage of elongation not less than 1 200 000 ÷ the tensile strength in pounds per square inch, and a percentage of reduction of area not less than 2 400 000; the tensile strength in pounds per square inch. In determining the ductility the elongation shall be measured after breaking on an original length of ten times the shortest dimension of the test piece, in which length must occur the curve of reduction from stretch on both sides of the point of fracture. Tensile tests.

19. Finished bars must be free from injurious flaws or cracks and must have a workmanlike finish, and round or square test pieces cut therefrom when pulled asunder shall have a reduction of area at the point of fracture, as above specified. Finish and reduction of area on finished bars.

TABLE No. 1.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen.	Pounds per Square Inch at		Per Cent of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches.	Reduction.			
6669	Comp'n Steel.	¾″□	48.570	72.560	24.75	48.70	Hom.	.....	Accepted.
6669	"	"	47.890	71.660	24.75	43.20	"	.....	"
6667	"	"	47.640	70.160	21.50	45.10	"	.....	"
6667	"	"	48.560	72.250	25.00	49.30	"	.....	"
6664	"	"	45.750	68.400	26.50	59.90	"	.....	"
6664	"	"	46.140	68.700	25.00	57 50	"	.....	"
6662	"	"	47.370	72.250	25.00	54.80	"	.....	"
6662	"	"	47.370	71.770	27 50	56.50	"	.....	"
6652	"	"	43.000	70.050	24.25	54.10	"	.....	"
6652	"	"	43.540	68.540	25.75	57.60	"	.....	"
6996	"	"	49.390	73.220	26.25	50.90	"	180° flat.	Rejected.
6996	"	"	48.560	71 800	26.25	54.50	"	" "	"
6982	"	"	46.000	69.250	27.00	58.80	"	" "	Accepted.
6984	"	"	45.820	71.360	26.25	57.80	"	" "	"
6994	"	"	46.150	69.960	26.25	57.50	"	" "	"
6985	"	"	47.700	70.940	27.00	56.20	"	" "	"
6983	"	"	47.250	70.170	25.00	60.40	"	" "	"
6981	"	"	45.820	67.300	26.75	59.20	"	" "	"
6992	"	"	45.350	68.020	26.00	54.90	"	" "	"
6986	"	"	48.430	70.700	27.50	58.80	"	" "	"
6987	"	"	44.800	67.800	25.00	45.50	"	" "	"
6990	"	"	45.300	65.600	26.00	62.10	"	" "	"
6960	"	"	46.500	71.300	24.25	53.50	"	" "	"
6849	"	"	46.200	68.530	26.50	61.30	"	" "	"
6850	"	"	45.610	70.080	30.00	58.00	"	" "	"

TABLE No. 2.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength	Stretch in 8 inches.	Reduction.			
6850	Comp. Steel.	¾″°	45.910	69.570	28.75	61.10	Hom.	180° flat.	Accepted.
6846	"	"	45.100	68.460	24.50	50.50	"	" "	"
6844	"	"	44.800	69.000	26.25	52.80	"	" "	"
6845	"	"	44.860	68.340	27.00	57.50	"	" "	"
6847	"	"	46.260	68.580	26.75	56.80	"	" "	"
6848	"	"	46.030	69.160	25.00	53.30	"	" "	"
6696	"	"	47.440	72.100	26.50	58.60	"	" "	"
6698	"	"	48.840	71.580	23.50	45.70	"	.....	"
6698	"	"	49.070	72.160	23.50	42.50	"	.....	"
6663	"	"	48.340	72.090	26.25	55.60	"	.....	"
6663	"	"	47.750	71.870	26.50	57.40	"	.....	"
6692	"	"	49.170	72.450	24.50	41.60	"	.....	"
6692	"	"	49.170	72.160	25.75	38.40	"	.....	"
6688	"	"	47.030	69.000	22.50	42.30	"	.....	"
6688	"	"	47.170	69.340	24.00	42.00	"	.....	"
6674	"	"	46.290	67.940	26.75	50.00	"	.....	"
6674	"	"	45.500	67.850	26.50	53.73	"	.....	"

TABLE No. 3.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches..	Reduction.			
257.705	Tension Steel.	¾″	45.290	68.260	26.5	59.7	Silky Cup.	180°	Accepted.
257.722	"	"	48.350	67.640	26.5	57.2	"	"	"
257.725	"	"	47.450	66.620	28.6	60.4	"	"	"
257.933	"	"	43.720	65.580	27.8	58.3	"	....	"
258.003	"	"	45.290	66.450	26.8	58.0	"	....	"
258.007	"	"	46.770	66.160	27.9	58.6	"	....	"
237.232	"	"	43.360	63.920	27.3	58.8	"	180°	"
257.236	"	"	46.950	66.460	26 2	54.8	"	"	"
257.267	"	"	44.110	64.820	25.4	58 0	"	"	"
257.309	"	"	46.090	66.910	26.9	52.0	"	....	"
257.315	"	"	47.350	69.900	26.9	62.2	"	....	"
257.318	"	"	47.350	64.550	28.6	63.1	"	180°	"
257.350	"	"	41.880	62.040	30 0	61.2	"	"	"
257.383	"	"	42.220	64.680	27.9	55.0	"	"	"
257.407	"	"	41.100	62.090	29.0	64.2	"	"	"
257.410	"	"	42.110	64.280	26.6	62.1	"	"	"
257.131	"	"	42.540	67.360	26.6	59.1	"	"	"
257.135	"	"	45.870	70.460	27.4	59.9	"	"	"
257.149	"	"	43.780	66.110	29.7	63.1	"	"	"
257.153	"	"	42.994	65.490	28.3	58.0	"	"	"
257.156	"	"	42.110	63.605	30.2	64.4	"	"	"
257.167	"	"	44.010	66.460	27.5	62.0	"	"	"
257.170	"	"	43.560	65.800	28.3	62.0	"	"	"
257.204	"	"	46.950	70.010	25.6	58.5	"	"	"
257.211	"	"	43.750	65.940	28.0	61.1	"	"	"

TABLE No. 4.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen. Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches.	Reduction.			
258.538	Tension Steel.	¾	43.250	64.980	28.4	55.8	Silky Cup.	180°	Accepted.
258.544	"	"	49.780	66.530	25.8	57.5	"	"	"
258.581	"	"	48.660	67.010	26.1	56.5	"	"	"
258.586	"	"	47.890	68.580	26.1	51 6	"	"	"
258.597	"	"	45.410	66.880	26.5	55.6	"	"	"
258.605	"	"	44.400	64.860	27.0	58 3	"	"	"
258.614	"	"	42.890	65 940	27 3	59.5	"	"	"
258.626	"	"	43.010	64.520	28.8	58.1	"	"	"
258.632	"	"	42 550	64.400	30.0	63.0	"	"	"
258.756	"	"	45.300	66 580	26.3	54.1	"	"	"
258.758	"	"	46.240	66.480	26.9	54.8	"	"	"
257.413	"	"	42.220	64.010	27.5	62 2	"	"	"
257.540	"	"	44.140	67.540	26.7	58.4	"	"	"
257.550	"	"	46.800	69 660	18.9	56.9	"	....	"
257.550	"	"	46.920	70.960	26.7	44.7	"	....	"
257.554	"	"	44.140	67.540	27.1	54.9	"	180°	"
257.561	"	"	44.010	64.680	28.8	56.8	"	"	"
257.576	"	"	45.300	65.430	28 7	62.1	"	"	"
257.584	"	"	47.230	67.300	27.6	61.3	"	"	"
257.590	"	"	44.630	65.550	24 4	58.3	"	"	"
257.634	"	"	44.610	65.540	27.2	62.3	"	"	"
257.645	"	"	46.090	68.660	27.8	60.4	"	"	"
257.648	"	"	47.400	67.850	26.9	56.6	"	"	"
257.654	"	"	50.420	69.580	24.8	54.2	"	"	"
257.694	"	"	45.620	68.440	26.6	58.6	"	"	"

TABLE No. 5.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen. Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Strength in 8 inches.	Reduction.			
335	9 x $\frac{3}{8}$" pl.	1.015 x .402	40.690	68.630	30.00	38.48	Silky cup.	180° on self.	Accepted.
336	10 x $\frac{3}{4}$" pl.	1.015 x .760	37.850	62.740	26.87	55.26	Silky cup.	" "	"
338	12 x $\frac{1}{2}$" pl.	1.020 x .510	49.980	76.320	24.00	35.36	Ang. silky.	" "	"
331	12 x $\frac{1}{2}$" pl.	1.018 x .505	42.610	66.150	25.00	56.10	Ang. silky.	" "	"
350	16 x $\frac{3}{8}$" pl.	1.000 x .753	43.750	66.060	27.50	51 44	Ang. silky.	" "	"
394	$17\frac{1}{2}$ x $\frac{13}{16}$" pl.	1.020 x .791	37.180	63.710	28.12	50.05	Silky cup.	" "	"
398	$17\frac{1}{2}$ x $\frac{13}{16}$" pl.	1.023 x .796	34.270	62.640	23.75	37.84	Silky cup.	" "	"
332	$19\frac{1}{4}$ x $\frac{1}{2}$" pl.	1.025 x .500	41.170	64.400	25.00	52.64	Silky cup.	" "	"
311	20 x $\frac{7}{16}$" pl.	1.030 x .430	41.100	63.230	25.00	54.74	Silky cup.	" "	"
593	20 x $\frac{9}{16}$" pl.	1.02 x 0.56	43.830	67.140	25.00	51.23	Silky cup.	" "	"
611	20 x $\frac{9}{16}$" pl.	1.01 x 0.57	45.140	67.270	25.00	53.87	Part cup and silky.	" "	"
610	20 x $\frac{11}{16}$" pl.	1.01 x 0.68	46.450	68.580	25.62	52.29	Part cup and silky.	" "	"
612	20 x $\frac{11}{16}$" pl.	0.99 x 0.68	46.210	62.600	25.62	53.66	Silky cup.	" "	"
578	20 x $\frac{11}{16}$" pl.	1.01 x 0.68	40.300	62.550	22.50	47.15	Ang. and silky.	" "	"
579	20 x $\frac{11}{16}$" pl.	.99 x 0.69	43.870	70.100	27.50	50.00	Silky cup.	" "	"
343	$21\frac{1}{2}$ x $\frac{5}{16}$" pl.	1.055 x .330	49.990	68.950	23.12	37.37	Silky cup.	" "	"
334	24 x $\frac{1}{2}$" pl.	1.025 x .501	41.680	68.370	24.75	47.50	Silky cup.	" "	"
399	30 x $\frac{1}{2}$" pl.	1.03 x 0.48	52.300	64.920	22.50	52.83	Silky cup.	" "	"
398	30 x $\frac{9}{16}$" pl.	1.015 x .548	44.950	62.930	23.75	44.50	Silky cup.	" "	"
406	30 x $\frac{9}{16}$" pl.	1.02 x 0.56	44.790	62.980	25.00	55.21	Silky cup.	" "	"
407	30 x $\frac{9}{16}$" pl.	1.00 x 0.56	39.980	63.120	26.25	52.28	Silky cup.	" "	"
390	30 x $\frac{9}{16}$" pl.	1.02 x 0.57	42.510	66.550	23.75	51.01	Silky cup.	" "	"
368	29 x $\frac{5}{8}$" pl	1.020 x .597	48.610	69.460	25.62	49.86	Silky cup.	" "	"
396	30 x $\frac{9}{16}$" pl.	.992 x .557	45.070	70.590	26.87	42.50	Silky cup.	" "	"

TABLE No. 6.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen. Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches.	Reduction.			
411	30 x $\frac{5}{8}$" pl.	1.01 x 0.61	39.900	68.410	23.12	45.13	Ang. and silky.	180° on self.	Accepted.
412	30 x $\frac{7}{8}$" pl.	1.05 x 0.86	41.750	68.280	24.37	42.01	Silky cup.	" "	"
415	30 x $\frac{5}{8}$" pl.	1.04 x 0.63	45.390	69.230	24.37	44.15	Silky cup.	" "	"
416	30 x $\frac{5}{8}$" pl.	1.01 x 0.61	43.260	69.110	22.50	39.90	Irreg. silky cup.	" "	"
400	30 x $\frac{5}{8}$" pl.	1.03 x 0 61	39 410	66.700	21.37	36.84	Ang. and silky.	....	"
401	30 x $\frac{5}{8}$" pl.	1.04 x 0.59	49.390	69.380	28.75	48.55	Silky cup.	....	"
402	30 x $\frac{5}{8}$" pl.	1.04 x 0.62	43 010	68.200	23.12	55.27	Silky cup.	....	"
403	30 x $\frac{5}{8}$" pl.	1.02 x 0.61	40.320	68.540	24.37	37.01	Irreg., ang. and silky.	....	"
389	30 x $\frac{5}{8}$" pl.	1.020 x .613	39.190	67.020	27.50	48.70	Irreg., ang. and silky.	....	"
393	30 x $\frac{5}{8}$" pl.	.995 x .613	42.630	68.050	24.	47.04	Irreg., ang. and silky.	....	"
395	30 x $\frac{5}{8}$" pl.	1. x .607	42.180	69.200	26.87	48.10	Irreg , ang. and silky.	....	"
397	30 x $\frac{5}{8}$" pl.	1.007 x .610	39.240	69.200	27.50	47.59	Irreg , ang. and silky.	....	"
359	30 x $\frac{5}{8}$" pl.	1.005 x .623	42.960	66.460	28.75	50.24	Silky cup.	....	"
360	30 x $\frac{5}{8}$" pl,	1.002 x .592	40.350	68.270	20.63	34.89	Silky ang.	180° on self.	"
404	30 x $\frac{5}{8}$" pl.	1.03 x 0 62	44.380	67.840	25.62	38.31	Ang. and silky.	" "	"
372	30 x $\frac{5}{8}$" pl.	1.020 x .626	43.700	65.780	26.87	41.86	Ang. and silky.	" "	"
382	30 x $\frac{5}{8}$" pl.	1.014 x .616	44.830	68.520	21.87	45.1	Ang. and silky.	" "	"
372	30 x $\frac{5}{8}$" pl.	1.003 x .624	41.550	63.280	25.	40.57	Ang. and silky.	" "	"
377	30 x $\frac{5}{8}$" pl.	1.035 x .631	41 350	66.620	26.25	46.50	Ang. and silky.	" "	"
383	30 x $\frac{11}{16}$" pl.	1.026 x .660	45.190	62.770	26.87	44.84	Ang. and silky.	" "	"
370	30 x $\frac{3}{4}$" pl.	1.023 x .751	38.010	70.670	23.12	32.29	Ang. and silky.	" "	"
348	30 x $\frac{3}{4}$" pl.	1.040 x .727	42.720	69.840	25.62	43.63	Silky cup.	" "	"
367	30 x $\frac{13}{16}$" pl.	1.035 x .795	41 920	65.620	25.62	40.93	Silky cup.	" "	"
378	30 x $\frac{7}{8}$" pl.	1.040 x .852	42.770	69.300	21.87	42.14	Silky cup.	" "	"
408	30 x $\frac{7}{8}$" pl.	1.03 x 0.86	37.390	68.200	27.50	47.04	Silky cup.	" "	"
409	30 x $\frac{7}{8}$" pl.	1.03 x 0.86	39.440	69.140	26.87	49.05	Silky cup.	" "	"

TABLE No. 7.

TESTS OF STEEL BY THE PHŒNIX BRIDGE COMPANY FOR THE CINCINNATI AND COVINGTON ELEVATED RAILWAY, TRANSFER AND BRIDGE COMPANY.

Heat.	Material.	Section of Test Specimen. Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches.	Reduction.			
259.479	7 x 1 1/8"	.71 x 1.14	39.890	61.190	30.7	59.12	Silky.	.....	Accepted.
258.368	7 x 1 1/16"	.72 x 1.09	43.970	68.060	28.6	51.10	Silky dull.	... .	"
258.368	7 x 1 1/16"	.72 x 1.10	44.350	67.420	27.8	48 6	Silky.	.....	"
257.634	7 x 1 1/16"	.71 x 1.10	45.530	66.800	27.8	47.54	Silky.	.....	"
257.540	7 x 1"	.72 x 1.04	48.590	66.790	26.7	33.3	Silky lam.	.....	"
257.350	7 x 1"	.75 x 1.01	37.550	59.030	29.4	48.2	Silky.	.....	"
257.648	7 x 1 1/16"	.71 x 1.07	47.240	67.820	28.5	48.7	Silky.	.....	"
257.383	7 x 1"	.75 x 1 01	44.750	61.070	26.9	42.8	Silky lam.	.....	"
257.550	7 x 1"	.75 x 1.04	48.530	70.090	27.1	46.33	Silky.	.....	"
257.225	7 x 1 1/16"	.72 x 1.09	45.550	69.290	25.6	41.6	Silky.	....	"
257.236	7 x 1 1/16"	.71 x 1.09	41.190	54.780	27.0	47.45	Silky lam.	.....	"
257.309	7 x 1 1/8"	.68 x 1.21	41.960	64.220	26.8	45.5	Silky pitt.	.....	"
257.315	7 x 1 1/4"	.60 x 1.29	45.360	66.740	26.0	39.4	Silky pitt.	.....	"
258.335	7 x 1 5/16"	.59 x 1.34	41.590	63.560	25.8	52.0	Silky lam.	.....	"
259.443	7 x 1 1/16"	.72 x 1.09	43.530	64.510	30.0	52.8	Silky lam.	.....	"
259.445	7 x 1 1/16"	.72 x 1.05	43.070	64.940	29.7	52.2	Silky lam.	....	"
259.808	7 x 1 1/16"	.71 x 1.09	38.620	62.430	29.7	55.8	Silky.	.....	"
261.138	7 x 1 1/16"	.72 x 1.08	42.190	64.570	27.1	41.1	Silky pitt.	.....	"
261.138	7 x 1 1/16"	.72 x 1.08	49.970	60.940	29.5	48.8	Silky lam.	....	"
261.287	7 x 1"	.72 x 1.05	40.540	65 660	28.5	48.0	Silky pitt.	.....	"
261.287	7 x 1"	.75 x 1.05	42.530	62.720	20.7	38.6	Silky lam.	.....	"
261.415	7 x 1"	.72 x 1.04	44.620	65.000	29.7	55.5	Silky lam.	.....	"
261.437	7 x 1 1/4"	.61 x 1.27	39.800	63.160	27.0	45.08	Silky.	.....	"
257.540	7 x 1"	.75 x 1.04	48.630	67.560	26.3	48.1	Silky.	.....	"

TABLE No. 8.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen. Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches.	Reduction.			
261.415	7 x 1"	.75 x 1.03	44.270	65.360	29.6	56.6	Silky.	.....	Accepted.
257.561	7 x 1"	.93 x 1.03	43.530	61.270	26.1	49.2	Silky pitt lam.	.....	"
261.386	7 x 1 1/16"	.72 x 1.09	41.980	64.240	28.3	51.2	Silky lam.	.....	"
257.267	7 x 1 1/16"	.73 x 1.08	41.330	64.090	29.6	56.06	Silky.	.....	"
258.605	7 x 1 1/16"	.71 x 1.01	43.860	64.650	27.6	48.3	Silky pitt.	.....	"
257.479	7 x 1 1/16"	.72 x 1.09	40.370	62.850	29.0	54 07	Silky pitt lam	.....	"
257.479	7 x 1 1/16"	.73 x 1.08	42.890	62.230	30.6	53.6	Silky.	.....	"
257.590	7 x 1 1/16"	.72 x 1.08	44.160	66.040	26.9	52.13	Silky lam.	.....	"
258.586	7 x 1 1/16"	.72 x 1 09	43.010	65.490	25.9	43.3	Silky pitt.	.....	"
258.586	7 x 1 1/16"	.71 x 1.09	42.140	65.490	27.5	45.5	Silky lam.	.....	"
259.215	7 x 1 1/8"	.65 x 1.14	42.840	66.370	24.5	42.9	Silky pitt.	.....	Rejected.
258.632	7 x 1 3/16"	.53 x 1.82	33.570	55.670	27.30	54.8	Silky pitt.	.....	Accepted.
257.634	7 x 1 3/16"	.65 x 1.21	44.650	67.100	27.9	45.3	Silky pitt.	.....	"
258.765	7 x 1 3/16"	.63 x 1.21	43.020	63.930	28.6	50.2	Silky lam.	.....	"
261.357	7 x 1 1/4"	.61 x 1.28	41.140	63.520	28.6	48.	Silky lam.	.....	"
261.156	7 x 1 3/8"	.51 x 1.42	39.310	58.780	14.	30.5	Silky lam.	.....	"
261.156	7 x 1 3/8"	.55 x 1.42	41.440	63.370	17.7	22.4	Gran.	.....	"
260.103	7 x 1 3/8"	.57 x 1.32	41.610	64.970	28.5	44 2	Gran.	.....	"
260.103	7 x 1 3/8"	.54 x 1.33	42.150	65.220	27.4	41.9	Silky gran.	.....	"
256.871	7 x 1 3/8"	.55 x 1.38	42.150	63.230	30.1	55.	Silky.	.....	"
261.153	7 x 1 3/8"	.55 x 1.43	41.940	62.380	22.5	54.6	Silky lam.	.....	"
261.153	7 x 1 3/8"	.55 x 1.41	41.230	60.850	27.	46.5	Silky pitt.	.....	"
261.133	7 x 1 3/8"	.53 x 1.41	39.790	61.100	27.1	51.1	Silky lam.	.....	"
261.198	7 x 1 7/16"	.53 x 1.46	42.680	61.400	30.4	52.5	Silky lam.	.....	"
259.808	7 x 1 7/16"	.53 x 1.47	41.020	62.300	30.	54.	Silky lam.	.....	"
257.309	7 x 1 5/8"	.05 x 1.65	38.680	64.430	25.8	50 26	Silky dull.	.....	"

TABLE No. 9.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen. Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches.	Reduction.			
257.149	7 x 1 5/8"	.49 x 1.63	40.690	64.060	26.	43.2	Silky.	.....	Accepted.
257.149	7 x 1 5/8"	.49 x 1.65	40.720	63.840	27.8	51.5	Silky.	.....	"
259.506	7 x 1 5/8"	.50 x 1.63	36.250	62.500	27.9	43.8	Silky.	.....	"
259.526	7 x 1 5/8"	.49 x 1.65	36.420	61.980	27.8	44.3	Silky pitt.	.....	"
261.527	7 x 1 5/8"	.51 x 1.69	42.280	60.240	28.6	56.1	Silky lam.	.....	"
258.756	7 x 1 13/16"	.50 x 1.83	40.050	64.980	26.6	46.5	Silky.	.....	"
261 324	7 x 1 13/16"	.51 x 1.84	39.280	63.060	30.7	50.8	Silky.	.....	"
261.318	7 x 1 13/16"	.49 x 1.83	39.740	61.810	27.2	49.1	Silky.	.....	"
261.114	7 x 1 13/16"	.49 x 1.84	38.430	63.020	27.	42.1	Silky.	.....	"
261.315	7 x 1 13/16"	.50 x 1.82	37.860	65.090	28.	37.8	Silky.	.....	"
259.410	7 x 1 13/16"	.57 x 1.84	35.690	64.040	26.90	37.5	Silky.	.....	"
259.354	7 x 1 13/16"	.56 x 1.85	38.990	62.580	29.60	52.90	Silky.	.....	"
259.153	7 x 1 13/16"	.51 x 1.85	39.570	62.560	32.2	56.6	Silky.	.....	"
257.590	7 x 1 13/16"	.58 x 1.82	39.170	64.690	27.4	39.15	Silky gran.	.....	"
257.413	7 x 2"	.51 x 2.02	36.880	62.400	30.9	39.6	Silky gran.	.....	"
257.554	7 x 2 1/16"	.51 x 1.96	41.850	63.610	32.1	48.6	Silky.	.....	"
259.654	7 x 2 1/16"	.51 x 1.96	39.360	60.980	30.0	43.5	Silky.	.....	"
257.960	7 x 2 1/16"	.51 x 1.95	40.530	62.010	30.5	50.3	Silky.	.....	"
21.541	8 x 1 7/16" pl.	Diameter .823	40.230	65.040	22.	44.53	Silky.	.....	"
21.561	8 x 1 7/16" pl.	" .825	39.280	64.630	25.75	47.12	Silky.	.....	"
21.553	8 x 1 7/16" pl.	" .815	40.830	71.400	27.	53.63	Silky.	.....	"
21.525	8 x 1 7/16" pl.	" .812	40.750	71.260	27.5	54.12	Silky.	.....	"
21.533	8 x 1 7/16" pl.	" .832	40.000	66.220	25.	51.41	Silky cup.	.....	"
21.529	8 x 1 7/16" pl.	" .825	40.400	64.540	25.75	52.6	Silky.	.....	"
21.549	8 x 1 7/16" pl.	" .820	41.380	65.330	25.75	48.23	Silky cup.	.....	"

TABLE No. 10.

Test of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Heat.	Material.	Section of Test Specimen. Diameter. Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture.	Bending Test.	Accepted or Rejected.
			Elastic Limit.	Ultimate Strength.	Stretch in 8 inches.	Reduction.			
21 557	8 x 1½" pl.	.825	42.280	69.400	22.5	43.16	Silky.	.....	
21 599	8 x 1 13/16" pl.	.812	38.630	65.280	25.	45.41	Silky.	.....	
21 603	8 x 1 13/16" pl.	.814	41.600	65.340	27.5	48.	Silky.	.....	
21 619	8 x 1 13/16" pl.	.815	40.640	64.600	25.5	49.36	Silky.	.....	
21 591	8 x 1 13/16" pl.	.812	42.290	67 600	24.	42.62	Silky.	.....	
21 598	8 x 1 13/16" pl.	.825	40.590	63.600	25.	47.98	Silky.	.....	
21 607	3 x 1⅞" pl.	.830	41.210	69.300	26.25	47.76	Silky.	.....	
	Rivet Steel.	¾" round.	37.440	64.610	29.	47.	Cup silky.	180° flat.	
	"	"	41.990	61.800	28.75	50.9	Cup silky.	"	
	"	"	40.130	65.590	28.	42.8	Irregular silky.	"	
	"	"	39.170	64.510	30.	61.2	Cup silky.	"	
	"	"	37.840	64.670	29.25	51.3	Cup silky.	"	
	"	"	38.400	64.330	30.	62.0	Irregular silky.	"	
	"	"	37.620	63.110	31.75	63.	Irregular silky.	"	
	"	"	38.810	63.920	28.75	59.5	Irregular silky.	"	
	"	"	36.750	66.400	27.25	46.8	Irregular silky.	"	
	"	15/16" round.	37.890	63.620	30.5	56.3	Cup silky.	"	
	"	"	38.090	62.500	31.	56.5	Cup silky.	"	

TABLE No. 11.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Link Plate.	Test Pieces; length = 8 inches.	Pounds per Square Inch at		Per Cent. of		Fracture and Remarks.
		Elastic Limit.	Ultimate.	Reduction.	Elongation.	
These Link Plates varied from 60" x 48" x 2½" to 78" x 25" x 2".	2.5 x ¾ inches.	.....	59.730	30.8	26.25	Irregular. Rejected.
	2.5 "	29.300	60.350	30.9	30.00	Irregular. Rejected.
	1 inch round.	31.720	66.240	13.0	13.00	Granular. Rejected.
	1 "	30.570	68.150	27.7	21.25	Granular. Rejected.
	1 "	34.650	66.880	34.4	23.5	Irregular. Accepted.
	1 x 1 inch.	30.560	66.030	23.0	14.0	
	1 inch round.	30.000	64.450	14.4	14.75	Granular. Rejected.
	1 "	34.900	68.820	31.8	16.75	Bent cold twice and straightened under hammer. Accepted.
	1 "	34.650	66.860	34.4	23.5	Irregular silky.
	1 "	31.720	66.240	13.0	13.0	Irregular silky.
	1 "	30.570	68.150	27.7	21.4	Irregular silky.
	1 "	35.300	58.600	....	16.75	Cup.
	1 "	36.600	64.000	54.8	28.00	Cup.
	1 "	39.800	65.600	31.5	23.25	Irregular silky.
	1 "	31.660	61.006	51.1	28.5	Irregular silky.
	1 "	31.540	61.290	51.9	29.75	Cup.
	1 "	31.840	64.700	23.4	21.25	Angular silky.
	1 "	31.840	61.400	40.0	23.5	Irregular silky.
	1 "	30.060	67.640	15.4	16.25	Irregular granular.
	1 "	36.900	66.750	34.4	21.75	Irregular granular.
	1 "	30.000	64.450	14.4	14.75	Irregular granular.
	1 "	30.570	60.890	54.4	29.5	Cup silky.
	1 "	37.700	67.260	34.4	21.75	Irregulary silky. (Retest)
	1 "	34.900	68.020	31.8	16.75	Irregular silky.
	1 "	32.860	67.890	31.0	18.0	Irregular silky.
	1 "	31.840	59.490	52.9	31.25	Cup silky.
	1 "	33.410	65.700	44.2	27.25	Irregular silky.
	1 "	30.440	65.160	54.4	28.25	Cup silky.
	1 "	33.503	60.950	25.9	19.75	Irregular fine granular.
	1 "	38.570	60.760	53.8	28.75	Cup silky.

TABLE No. 12.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Link Plate.	Test Piece; length = 8 inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture and Remarks.
		Elastic Limit.	Ultimate	Reduction.	Elongation.	
These Link Plates varied from 60" x 48" x 2½ to 78" x 25" x 2".	1 inch round.	36.180	63.120	41.0	27.5	Angular silky.
	1 "	35.660	64.580	44.7	26.5	Angular silky.
	1 "	34.770	64.640	54.4	30.0	Cup silky.
	1 "	29.670	60.760	60.2	38.0	Cup silky.
	1 "	31.640	63.820	43.0	25.0	Angular silky.
	1 "	26.620	61.020	36.0	16.75	Cup silky.
	1 "	32.160	61.530	53.7	27.5	Cup silky.
	1 "	31.460	63.820	54.0	25.5	Cup silky.
	1 "	29.550	62.290	32.5	21.75	Irregular fine granular.
	1 "	29.420	57.000	62.8	31.75	Cup silky.
	1 "	29.170	62.540	54.4	29.5	Cup silky.
	1 "	30.190	64.570	38.4	26.25	Irregular silky.
	1 "	26.620	51.590	68.6	34.5	Cup silky.
	1 "	29.990	63.820	49.0	26.25	Cup silky.
	1 "	28.020	61.140	16.4	17.5	Irregular silky.
	1 "	27.180	65.230	34.1	22.0	Irregular silky.
	1 "	24.730	51.620	69.8	33.75	Cup silky.
	1.965 x 0.492	27.720	55.640	44.5	28.75	Angular silky.
	1 inch round.	26.580	62.780	23.9	19.25	Irregular granular.
	1 "	28.100	60.770	50.5	26.25	Cup silky.
	1 "	25.470	58.460	25.8	18.75	Irregular silky.
	1 "	27.260	61.780	29.0	18.00	Angular silky.
	1 "	24.840	61.460	54.6	26.5	Cup silky.
	1 "	29.020	61.510	35.8	24.25	Irregular silky.
	1 "	28.530	61.270	50.9	29.25	Angular silky.
	1 "	29.300	62.030	57.4	29.00	Cup silky.
	1 "	30.650	62.820	54.8	27.25	Cup silky.
	1 "	28.530	61.140	50.3	27.50	Cup silky.
	1 "	28.020	60.440	39.8	25.50	Angular silky.
	1 "	26.880	58.340	63.0	30.00	Cup silky.

TABLE No. 13.

Tests of Steel by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Link Plate.	Test Piece; length = 8 inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture and Remarks.
		Elastic Limit.	Ultimate.	Reduction.	Elongation.	
These Link Plates varied from 60" x 48" x 2½" to 78" x 25" x 2".	1 inch round.	28.530	61.140	50.3	27.5	Cup silky.
	1 "	28.020	60.440	39.8	25 5	Angular silky.
	1 "	26.880	58.340	63.0	30.0	Cup silky.
	1 "	28.020	57.450	62.1	31.25	Cup silky.
	1 "	24.260	60 280	51.6	25.00	Irregular silky.
	1 "	25.860	63.310	51.9	25.50	Angular silky.
	1 "	26.110	63.310	54.5	28.75	Cup silky.
	1 "	26.620	60.510	53.7	23.75	Cup silky.
	1 "	26.790	63.960	25.9	17.25	Irregular silky.
	1 "	26.114	63.552	50.3	25.25	Cup silky.
	1 "	26.114	61.400	53.2	31.5	Cup silky.
	1.25 "	33.190	63.990	44.68	27.5	Cup silky.
	1.16 "	35.740	64.310	37.68	26.5	Cup silky.

TABLE No. 14.

Tests of Steel and Iron Angles by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railway, Transfer and Bridge Company.

Size of Angle.		Test Piece; length = 8 Inches.	Pounds per Square Inch at		Per Cent. of Final		Fracture and Remarks.
			Elastic Limit.	Ultimate.	Reduction.	Elongation.	
5 x 3" 25 pounds.	Steel.	1 x 0.32"	39.430	71.290	52.9	21.25	Angular silky.
" 41 "	Steel.	1 x 0.55"	39.110	71.580	55.5	27.25	Cup silky.
" 49 "	Steel.	1 x 0.625"	38.650	69.650	46.4	26.00	Cup silky.
6 x 4" 42 "	Steel.	1 x 0.4"	39.440	71.105	54.3	23.75	Irregular silky.
" 44 "	Steel.	1 x 0.43"	39.010	69.860	52.7	23.75	Angular silky.
" 72 "	Steel.	1 x 0.74"	35.610	73.970	51.9	28.25	" "
" 72 "	Steel.	1 x 0.73"	36.930	72.910	53.4	27.75	" "
" 84 "	Steel.	1 x 0.85"	33.290	65.870	41.0	26.25	" "
" 84 "	Iron.	1 x 0.84"	34.170	63.060	60.8	31.25	" "
6 x 6" 57 "	Iron.	1 x 0.49"	26.120	46.920	28.1	20.50	Fibrous.
" 57 "	Iron.	1 x 0.50"	27.270	46.060	18.9	13.75	"
" 86 "	Iron.	1 x 0.71"	29.600	50.600	25.3	21.75	"
" 86 "	Iron.	1 x 0.70"	27.190	47.910	21.7	19.25	"
" 95 "	Iron.	1 x 0.86"	25.110	45.910	23.9	20.00	"
" 95 "	Iron.	1 x 0.86"	25.230	48.120	28.7	26.25	"
" 100 "	Iron.	1 x 0.88"	24.420	47.220	25.5	22.50	"
" 100 "	Iron.	1 x 0.90"	24.309	46.180	30.9	25.50	"
" 108 "	Iron.	1 x 0.98"	24.540	47.000	21.2	21.25	"
" 108 "	Iron.	1 x 0.97"	24 050	48.090	26.0	20.50	"
" 108 "	Iron.	1 x 0.98"	24.670	48.730	30.1	25.00	"

DISCUSSION.

F. C. Osborn, M. Am. Soc. C. E.—How was the strain in upper flange of top chord, due to weight of chord itself, taken care of? Were the pins placed below center of gravity of section, or were the bending strains subtracted from the compressive unit chord stresses as obtained from the formula?

William H. Burr, M. Am. Soc. C. E.—They were placed below the center of gravity, so that the bending from overweight was neutralized as nearly as practicable.

J. B. Johnson, M. Am. Soc. C. E.—How was the tension on the rivets prevented in the attachment of the floor beams to the posts?

Mr. Burr.—There were no rivets used in that connection; they were closely fitted turn bolts. If you will notice the detail drawings, you will find that the floor beams are continuous with the roadway floor bracket. Its upper chord passes through a horizontal slit cut in the post, so that there is no possibility of tension on those turned bolts. While there is no objection to tension in such bolts, you will see that the detail used perfectly avoids it.

C. L. Strobel, M. Am. Soc. C. E.—Is the first joint of the top chord, at top of the inclined end post, a pin-bearing joint?

Mr. Burr.—The joint at the upper end of the end post is a pin-bearing joint.

Mr. Strobel.—Do you consider that there is any necessity for making this joint a pin-bearing joint? Would not an abutting joint have been more satisfactory and more consistent, in view of the fact that the other top chord joints are abutting joints?

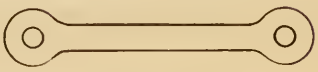
Mr. Burr.—Such a construction would be perfectly proper, but no more satisfactory than that used. A pin-bearing joint was avoided at the bend in the upper chord in order to obviate the use of steel links. We did not want any more of them than absolutely necessary.

Mr. Osborn.—I don't think Professor Burr has answered the first part of my question. If the pin was placed in the center of gravity, of course there is no bending on account of eccentricity, but the weight of the member is still there.

Mr. Burr.—The specifications have provided for that. The small fraction of overweight bending remaining uncared for, and arising from the fact that it is not practicable to place the pin centers at a uniform distance below centers of gravity, was considered in fixing the greatest allowed compression in the upper chord. With the 30-inch depth of chord used, the overweight bending is very small.

TABLE No. 15.

Tests of full-size Steel Eye-bars by The Phœnix Bridge Company for the Cincinnati and Covington Elevated Railroad, Transfer and Bridge Company.



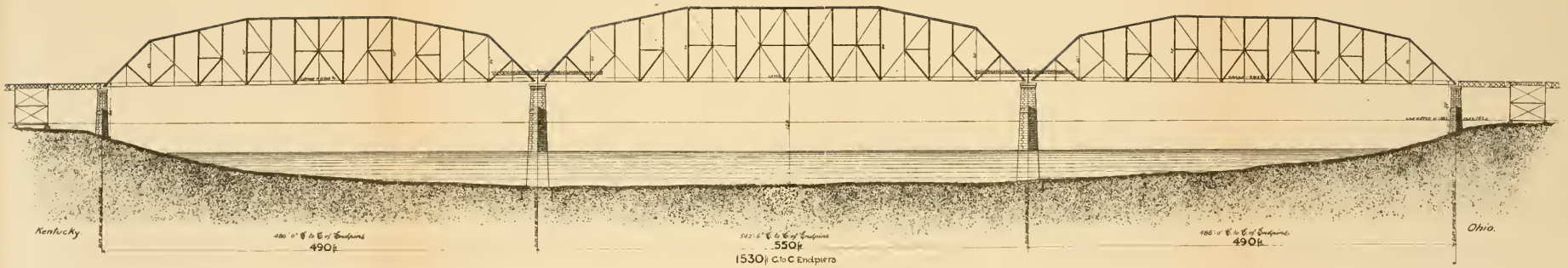
All pin-holes $6\frac{15}{16}''$ diameter.

Size of Bar.	Length c to c Pin-holes.	Per Square Inch of Original Section		Per Cent. Final			Fracture.	Accepted or Rejected.
		Elastic Limit.	Ultimate.	Stretch. in.	Stretch. in 12″.	Contraction.		
					Per Cent.	Per Cent.		
7 x $1\frac{5}{8}''$	26′ 7.9″	40.756	63.085	24 ft. 14.8 p. c.	$35\frac{8}{10}$	$48\frac{4}{10}$	Silky.	Accepted.
7 x $1\frac{3}{8}''$	20′ 11.9″	41.000	64.130	18 ft. 20.5 p. c.	$34\frac{9}{10}$	$41\frac{3}{10}$	Silky. Broke in head.	"
7 x $1\frac{13}{16}''$	27′ 9.5″	36.930	60.700	25 ft. 12 p. c.	$18\frac{9}{10}$	$13\frac{9}{10}$	Gran. and crys. Granular, 25 %.	"
7 x $1\frac{1}{16}''$	27′ 9.5″	38.743	63.370	28 ft. 15.7 p. c.	$38\frac{1}{5}$	$43\frac{6}{10}$	Silky, 75 %. Silky center, granular.	"
7 x $1\frac{1}{16}''$	27′ 9.5″	39.740	63.200	25 ft. 17.7 p. c.	$36\frac{2}{5}$	$45\frac{7}{10}$	Edges, ragged.	"
7 x $1\frac{1}{16}''$	27′ 9.5″	40.570	63.020	25 ft. 14.4 p. c.	$32\frac{1}{2}$	$44\frac{7}{10}$	Ragged, silky.	"
7 x $1\frac{13}{16}''$	27′ 9.6″	37.140	61.790	25 ft. 17.1 p. c.	$35\frac{2}{5}$	$43\frac{6}{10}$	Silky center, granular ragged edges.	"
7 x $1\frac{13}{16}''$	26′ 8″	36 048	59.290	24 ft. 17.6 p. c.	$42\frac{1}{2}$	$54\frac{6}{10}$	Center cupped silky. Edges granular.	"
7 x $1\frac{5}{8}''$	26′ 7.9″	39.380	62.940	24 ft. 20.1 p. c.	$36\frac{3}{5}$	$48\frac{1}{10}$	Silky, granular, ragged edge.	"
7 x $1\frac{1}{2}''$	26′ 8″	38.460	62.778	24 ft. 16.7 p. c.	35	$50\frac{3}{10}$	Silky center. Ragged gran. edges.	"
7 x $1\frac{13}{16}''$	26′ 8″	36.881	59.932	24 ft. 19.4 p. c.	40	$50\frac{6}{10}$	Silky cup. center. Ragged gran. edges.	"
7 x 2″	26′ 8″	39.080	62.240	24 ft. 13 3 p. c.	$40\frac{9}{10}$	$40\frac{7}{10}$	Silky cup. center. Ragged gran. edges.	"
7 x $2\frac{1}{16}''$	26′ 8″	37.500	53.880	24 ft. 7.0 p. c.	$8\frac{3}{10}$	Broke in head	Granular center. Finely crys. edges.	"
7 x $1\frac{5}{16}''$	26′ 7.6″	40.836	62.910	23 ft. 20.75 p. c.	40	56.16	Silky center, fine, granular on edges.	Rejected.
7 x $1\frac{1}{16}''$	26′ 7.6″	38.600	61.014	23 ft. 13.6 p. c.	37.5	52.7	Silky center, fine, granular on edges.	Accepted.
7 x $1\frac{1}{4}''$	26′ 7.6″	43.620	66.827	23 ft. 13.6 p. c.	17.5	....	Broke in head.	"
7 x 1″	26′ 7.6″	42.846	59.187	23 ft. 9.3 p. c.	11.66	....	Broke in head.	"
7 x 1″	26′ 7.6″	42.846	64.844	23 ft. 15.04 p. c.	37.5	57.4	Silky cup.	"
7 x $2\frac{1}{16}''$	26′ 7.6″	38.400	62.600	24 ft. 17.2 p. c.	38.3	55.1	Silky.	"
8 x $1\frac{13}{16}''$	23′ 10.8″ *B*	34.900	58.489	20 ft. 17.8 p. c.	37.5	45.8	60 % silky.	"
8 x $1\frac{7}{16}''$	34′ 6.5″ *B*	38.477	63.950	31 ft. 14.3 p. c.	35.8	51.2	Silky at 45 %.	"
8 x $1\frac{13}{16}''$	23′ 10.8″ *B*	33.337	58.656	20 ft. 21.7 p. c.	36.6	56.2	70 % silky cup.	"
8 x $1\frac{3}{16}''$	22′ 7.0″ *B*	34.060	56.200	20 ft. 16.4 p. c.	36.7	54.6	70 % silky cup.	"
8 x $1\frac{1}{4}''$	22′ 7.0″ *B*	44.800	72.750	20 ft. 17.5 p. c.	37.5	56.3	60 % silky. Broke in head.	"
7 x $2\frac{1}{2}''$	27′ 1.4″	40.230	59.520	25 ft. 8.96 p. c.	11.6	....	Fine granular.	Rejected.
7 x $1\frac{5}{8}''$	27′ 1.4″	39.160	64.850	25 ft. 18.26 p. c.	30.8	51.2	Silky gran. edges.	Accepted.
7 x $1\frac{13}{16}''$	27′ 1.4″	38.000	60.200	25 ft. 16.86 p. c.	33.33	56.6	Silky gran. edges. Broke in head.	Rejected.
7 x 2″	27′ 1.4″	39.860	55.370	25 ft. 7 p. c.	8.35	....	Coarse gran.	"
7 x $2\frac{1}{2}''$	27′ 1.4″	.....	38.900	.....	....	....	7″ pin used with $\frac{5}{16}$ bushing. Broke in head.	"
7 x 2″	27′ 1.4″	40.600	61.300	25 ft. 13.3 p. c.	39.1	48.7	Ragged, silky. Broke in head.	Accepted.
7 x $1\frac{13}{16}''$	27′ 1.4″	39.000	62.440	25 ft. 17.66 p. c.	32.5	....	Fine granular. Broke in head.	"
7 x $1\frac{5}{8}''$	27′ 1.4″	40.720	60.400	25 ft. 9.33 p. c.	13.33	....	Fine granular. Broke in head.	Rejected.
7 x $1\frac{5}{8}''$	27′ 1.4″	38.840	61.450	25 ft. 11.7 p. c.	16.66	....	Fine granular.	"
7 x $1\frac{3}{8}''$	27′ 1.4″	40.000	64.170	25 ft. 16.2 p. c.	33.3	45.4	Silky, crys. edges. Broke in head.	"
7 x $1\frac{3}{8}''$	24′ 11.9″	46.100	56.650	22 ft. 6.2 p. c.	6.66	....	Fine crys., coarse on edges	"
7 x $1\frac{5}{8}''$	27′ 1.4″	38.670	60.792	25 ft. 14.37 p. c.	34.2	4.52	Silky cup. center. 20 % gran. on edges.	Accepted.
7 x $1\frac{3}{8}''$	27′ 1.4″	36.200	57.670	25 ft. 14.4 p. c.	39.2	49.5	50 % silky cup. 50 % silky jagged.	"
7 x $1\frac{5}{8}''$	27′ 1.5″	39.680	61.670	25 ft. 14.8 p. c.	35.0	43.3	Silky cup. center; 25 % gran. edges.	"
7 x $1\frac{3}{8}''$	25′ 1.1″	38.689	62.660	24 ft. 15.85 p. c.	34.2	42.7	Silky center, gran. jagged edges.	"
7 x $1\frac{1}{16}''$	27′ 9.3″	42.070	69.980	26 ft. 11.45 p. 3.	24.58	46.2	25 % granular. 75 % silky cup.	"
7 x 1″	27′ 7.9″	45.680	72.160	25 ft. 15.79 p. c.	33.75	43.2	Silky ½ cupped.	"
7 x $1\frac{1}{16}''$	27′ 9.4″	42.690	67.300	26 ft. 14.2 p. c.	30.4	50.1	Silky cupped.	"
7 x $1\frac{3}{8}''$	20′ 11.9″	38.850	60.860	19 ft. 19.3 p. c.	35.0	43.4	Silky jagged center, 12 % gran. edges.	"
7 x $1\frac{5}{8}''$	26′ 8.0″	38.080	60.460	24 ft. 14.5 p. c.	35.0	39.5	55 % silky center. 45 % granular.	"
7 x $1\frac{13}{16}''$	26′ 8.0″	39.540	61.800	24 ft. 14.3 p. c.	39.1	47.2	Jagged, silky.	"
7 x 2″	26′ 8.0″	37.360	59.160	24 ft. 15.5 p. c.	36.6	50.3	Silky cupped.	"
7 x $2\frac{1}{16}''$	26′ 8.0″	36.800	50.400	24 ft. 15.6 p. c.	38.3	50.8	Silky ½ cupped.	"
7 x $1\frac{13}{16}''$	27′ 9.5″	38.780	50.000	26 ft. 15.5 p. c.	40.0	51.06	Silky cupped; 15% gran. edges.	"
7 x $1\frac{13}{16}''$	27′ 9.6″	38.900	62.270	26 ft. 16.1 p. c.	25.8	46.8	Silky cupped, 10 % fine gran. edges.	"

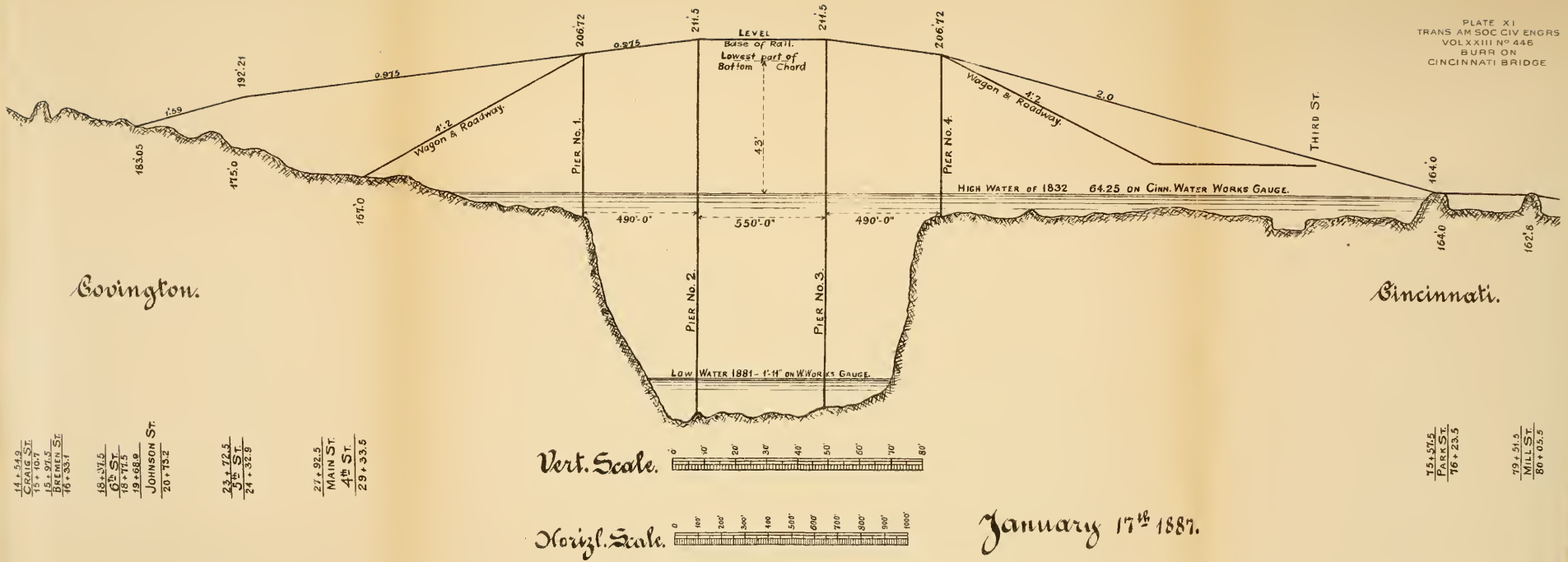
Reannealed.

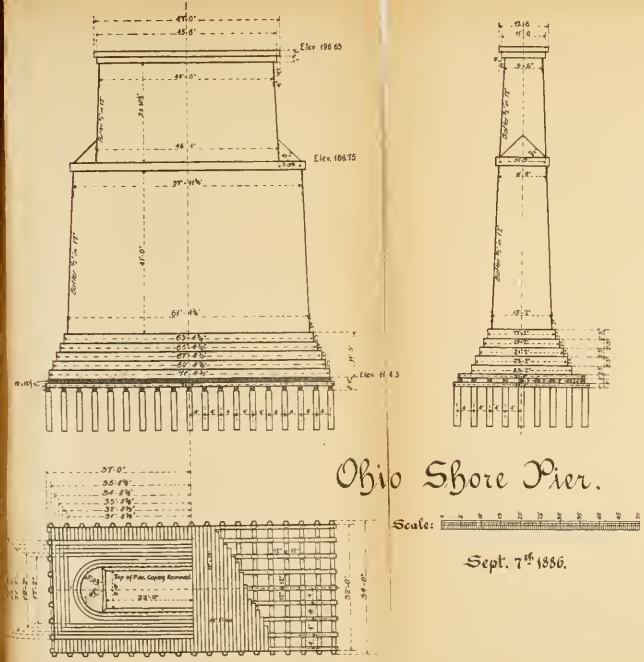
Reannealed.

Note.—"*B*" indicates Bessemer steel; all others are open hearth metal.



GENERAL ELEVATION
OF THE
RIVER SPANS - OHIO RIVER BRIDGE.
THE COVINGTON AND CINCINNATI ELEVATED RAILROAD
AND TRANSFER AND BRIDGE CO
DESIGNED, CONSTRUCTED & ERECTED BY
THE PHOENIX BRIDGE COMPANY
ENGINEERS & CONTRACTORS
PHOENIXVILLE PA
1888

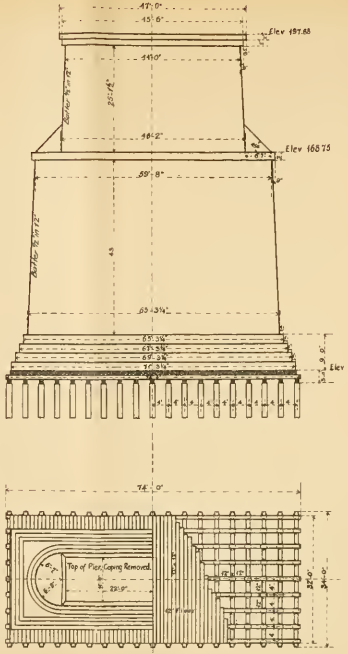

LEVEL
Base of Rail.
Lowest part of Bottom Chord
206.72
211.5
211.5
206.72
0.975
0.975
1:59
192.21
183.05
175.0
167.0
4:2
Wagon & Roadway
4:2
Wagon & Roadway
2.0
THIRD ST.
164.0
PIER No. 1.
PIER No. 2.
PIER No. 3.
PIER No. 4.
43'
HIGH WATER OF 1832 64.25 ON CINN. WATER WORKS GAUGE.
490'-0"
550'-0"
490'-0"
164.0
162.8
LOW WATER 1881 - 1'-11" ON W. WORKS GAUGE.
Covington.
Cincinnati.
14 + 54.9 / CRAIG ST. / 15 + 10.7
15 + 97.5 / BREMEN ST. / 16 + 33.1
18 + 71.5 / 6th ST. / 18 + 71.5
19 + 68.9 / JOHNSON ST. / 20 + 13.2
23 + 72.5 / 3rd ST. / 24 + 32.9
27 + 92.5 / MAIN ST. / 4th ST. / 29 + 33.5
75 + 57.5 / PARK ST. / 76 + 23.5
79 + 51.5 / MILL ST. / 80 + 05.5
Vert. Scale.
0 10' 20' 30' 40' 50' 60' 70' 80'
Horizl. Scale.
0 100' 200' 300' 400' 500' 600' 700' 800' 900' 1000'
January 17th 1887.

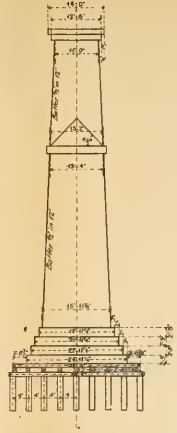


Ohio Shore Pier.

Scale:

Sept. 7th 1886.

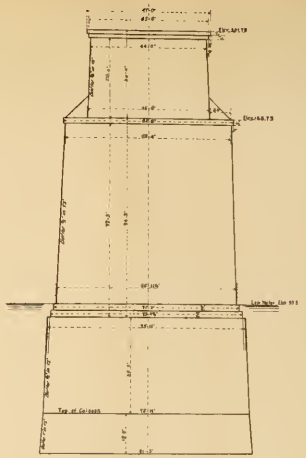


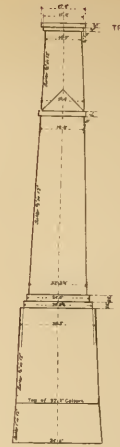


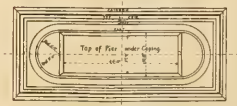
Kentucky Shore Pier.

Scale:

September 12th 1886.





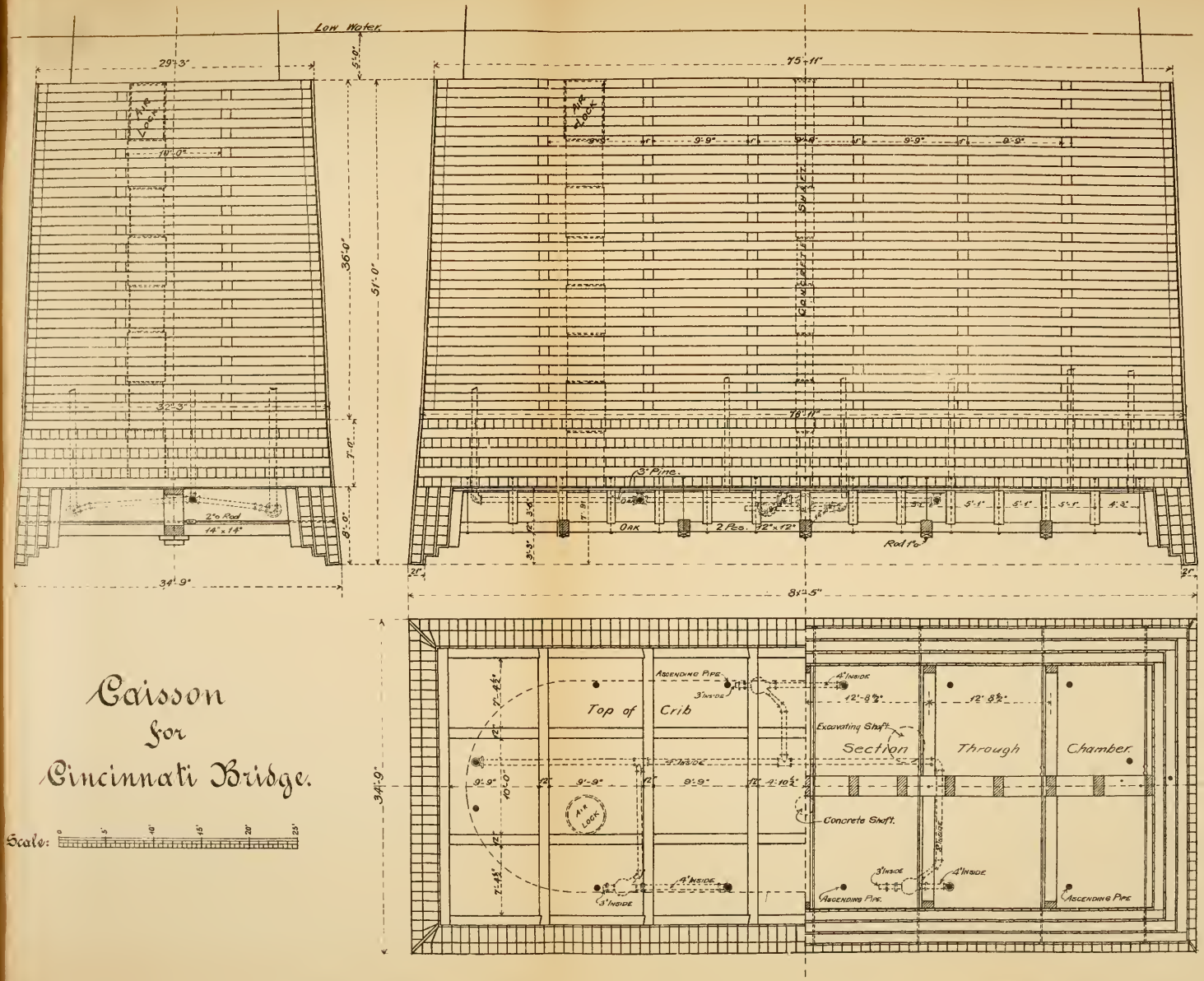


River Pier.

Scale:

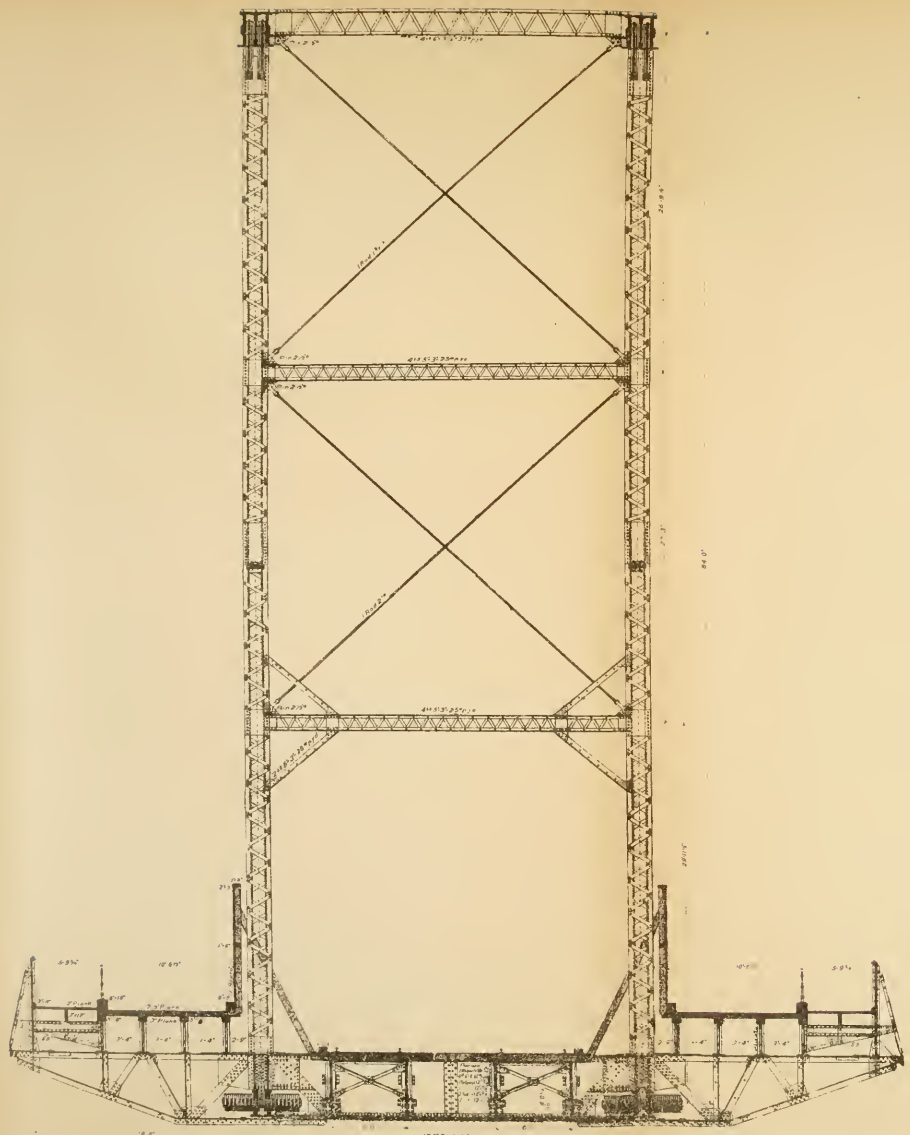
August 24th 1886.

PLATE XIII
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXIII No. 446
BURR ON
CINCINNATI BRIDGE.

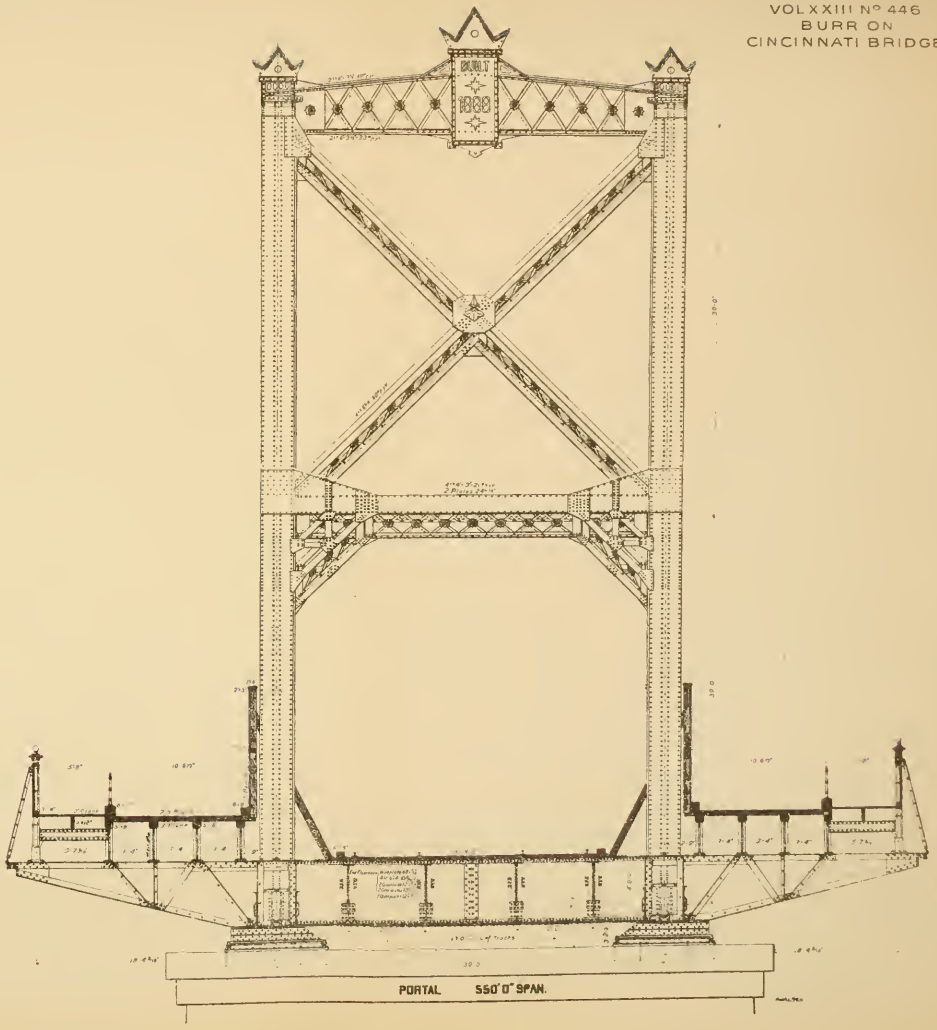


Caisson for Cincinnati Bridge.

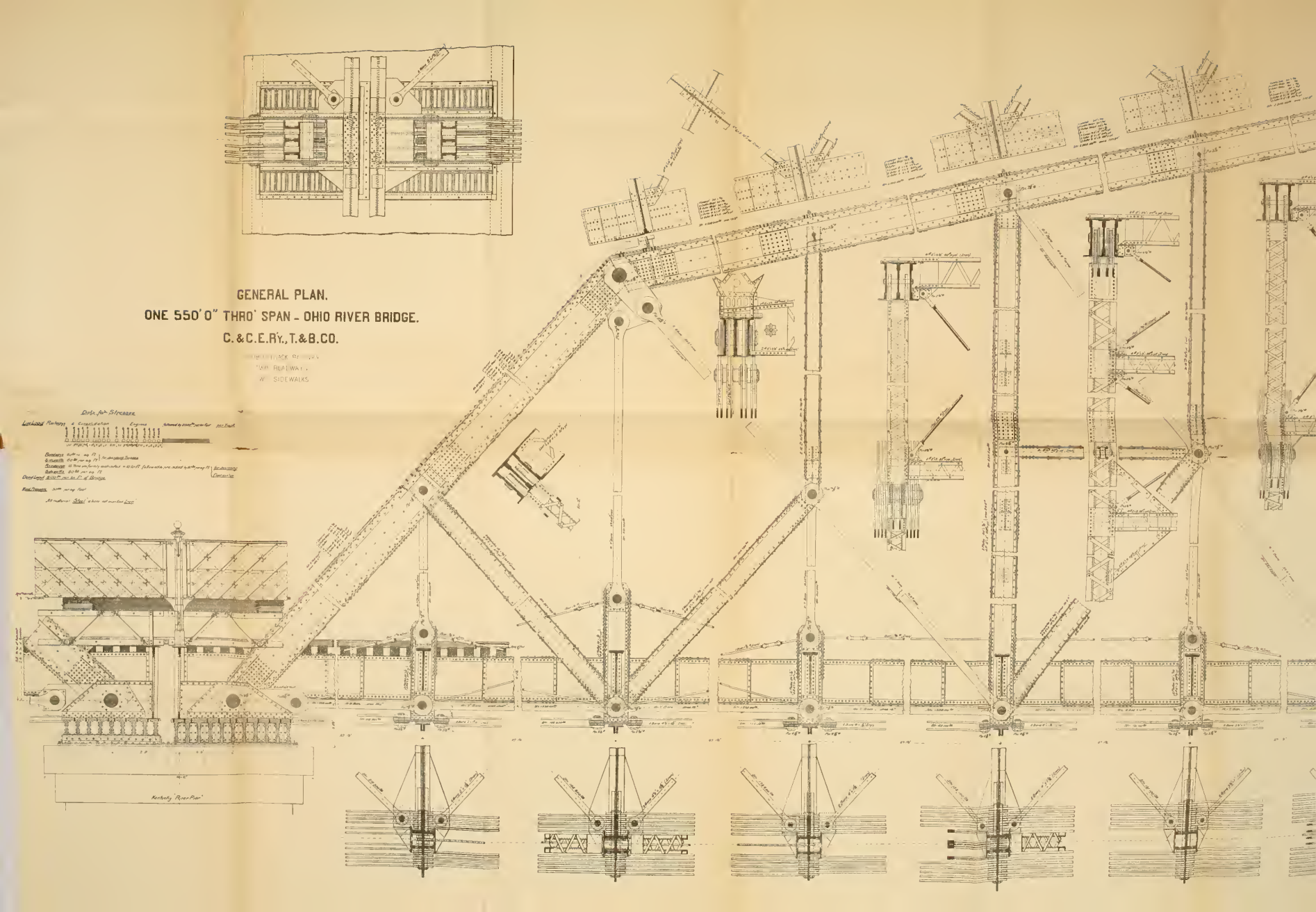
PLATE XIV
TRANS AM SOC CIV ENGRS
VOL XXIII Nº 446
BURR ON
CINCINNATI BRIDGE



CENTRE SECTION 550' 0" SPAN



PORTAL 550' 0" SPAN.


GENERAL PLAN.
ONE 550' 0" THRO' SPAN - OHIO RIVER BRIDGE.
C. & C. E. RY., T. & B. CO.
SIDEWALKS
Kentucky River Pier

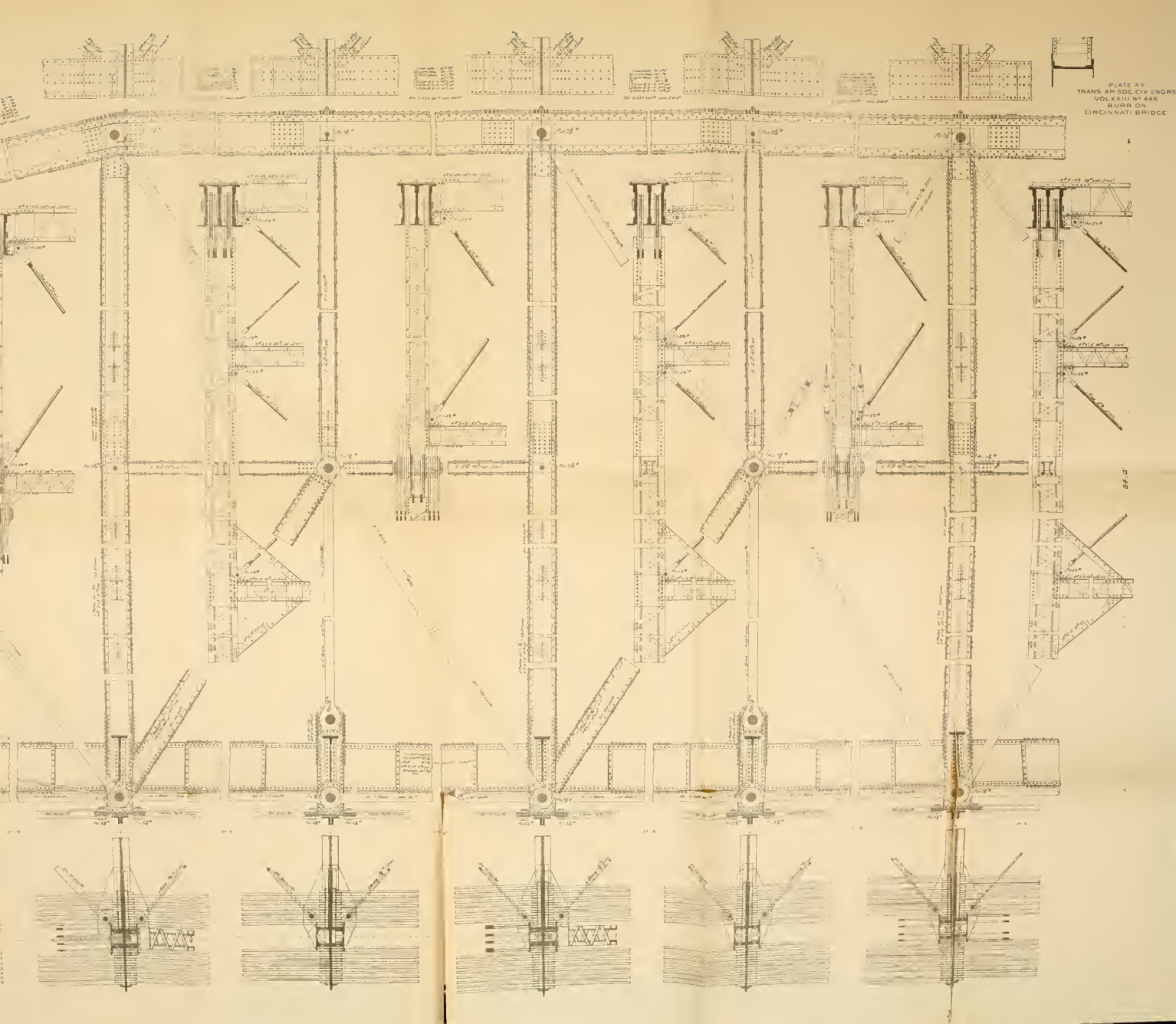
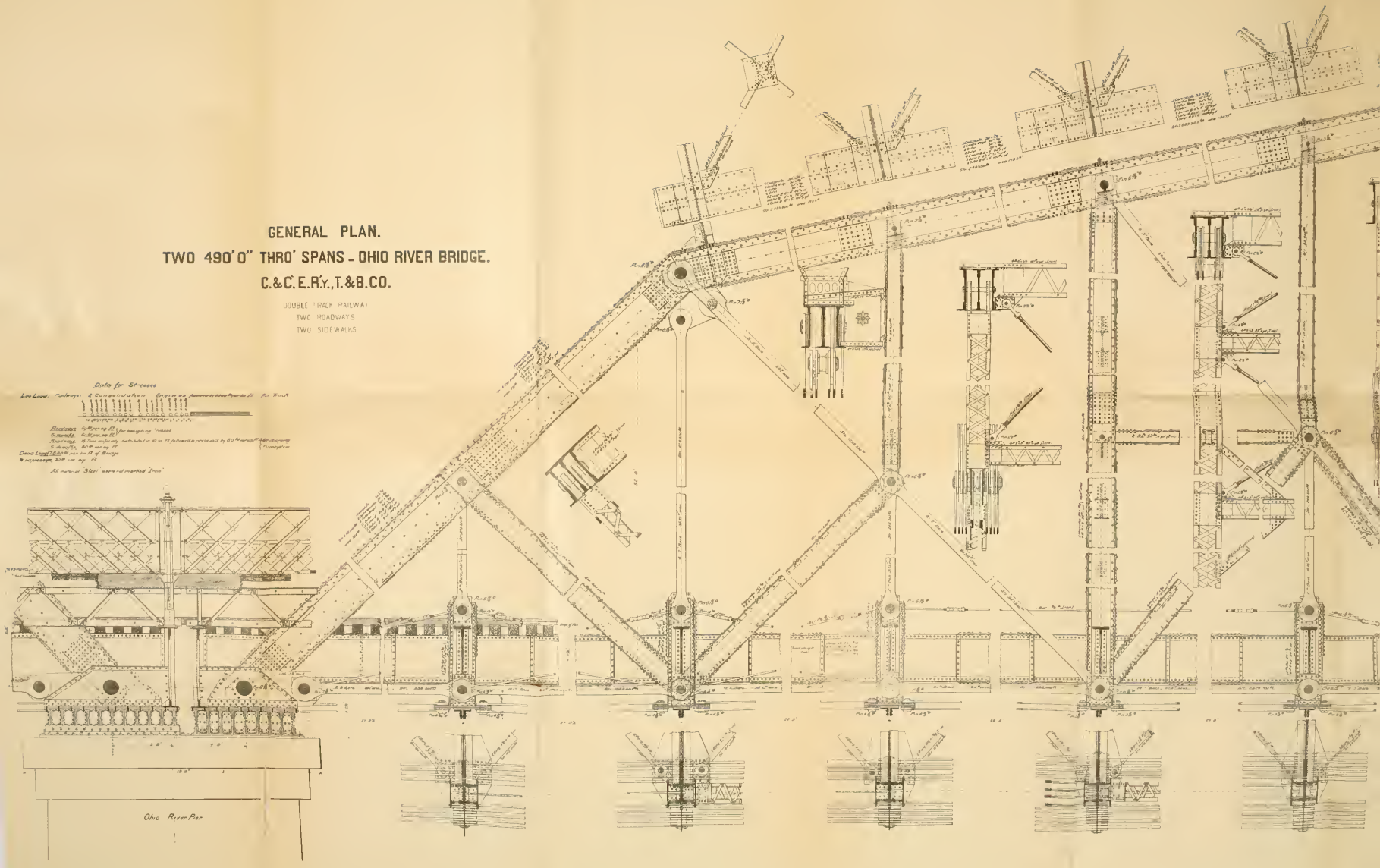


PLATE XV
TRANS AM SOC CIV ENGRS
VOL XXIII Nº 446
BURR ON
CINCINNATI BRIDGE


GENERAL PLAN.
TWO 490'0" THRO' SPANS - OHIO RIVER BRIDGE.
C.&C.E.R'Y.,T.&B.CO.
DOUBLE TRACK RAILWAY
TWO ROADWAYS
TWO SIDEWALKS
Data for Stresses
Ohio River Pier

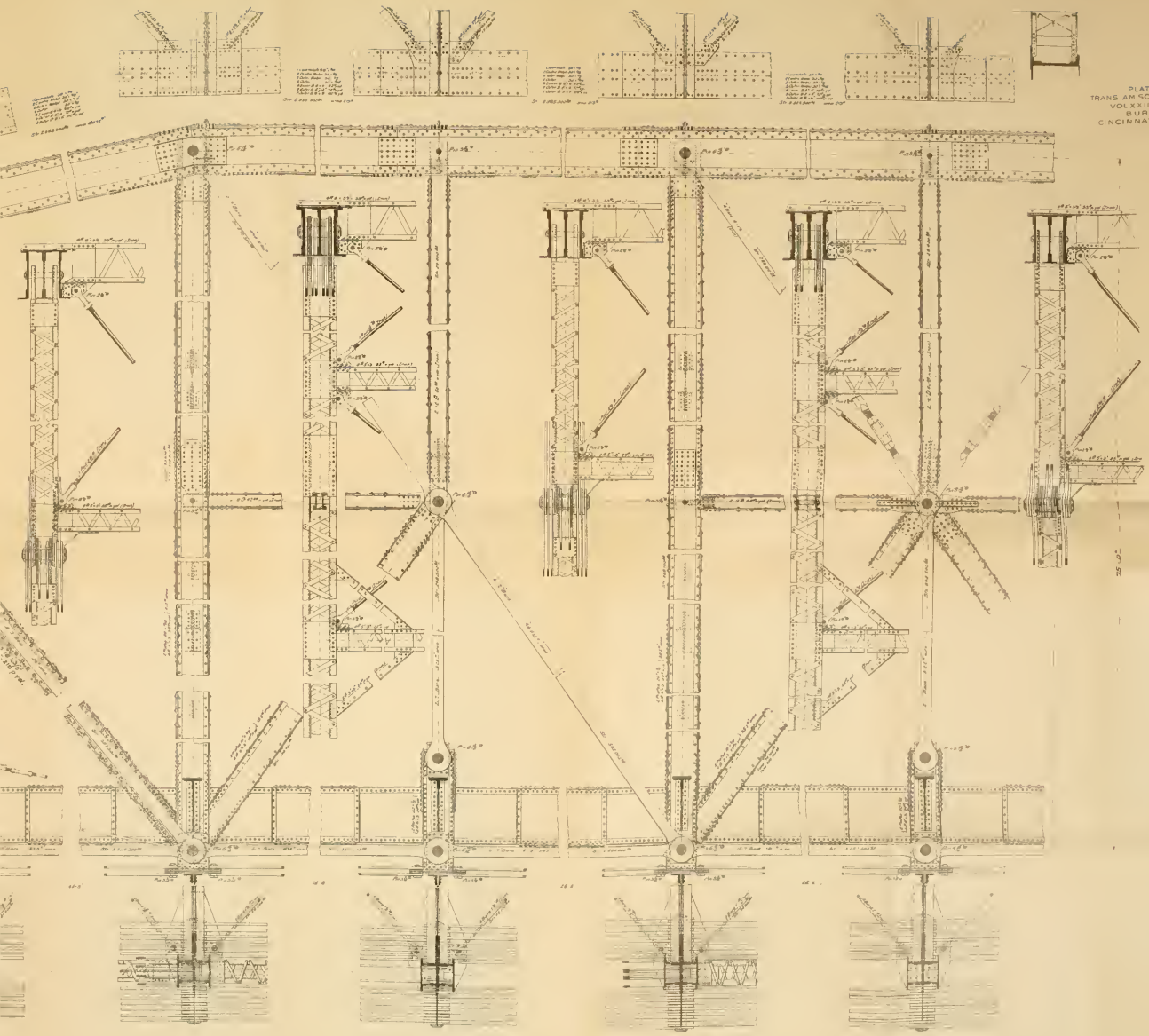


PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XIII. No. 446
BURR ON
CINCINNATI BRIDGE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

447.

Vol. XXIII.—August, 1890.

ONE WAY OF OBTAINING BRINE.

By CHARLES B. BRUSH, M. Am. Soc. C. E.

WITH DISCUSSION.

The remains of the oldest and largest salt works in this country are at Syracuse, N. Y. These works have depended on the brine obtained from wells sunk in the State Salt Reservation, which brine ranges from 63 to 73 per cent. saturation.

The very existence of these salt industries depends upon their obtaining an ample and satisfactory supply of brine from some source, and efforts to obtain such a supply have long been made by the State and private individuals.

It was not until the Tully Pipe Line Company, through its president, William B. Cogswell, M. Am. Soc. C. E., had made numerous exhaustive and extensive examinations in the vicinity of Syracuse, that results were obtained which promise to revive the salt business in that locality, and make it once more the leading industry of its kind in this country.

These investigations developed the fact that rock salt beds exist about 80 miles east of Warsaw and 20 miles south of Syracuse. Here are found two beds overlying each other, commencing about 1 200 feet below the surface. The first bed is 43 feet thick, then slate rock is encountered 25 feet thick, then the second bed of rock salt 54 feet thick. The ground at this point is about 400 feet above the Onondaga Lake at

Syracuse, and 420 feet below the Tully Lakes, which lie on the elevated plateau 3 miles further south.

Twelve wells have been sunk, in three groups, and work is progressing for eight additional wells. The groups are about 1 000 feet apart, and each group contains four wells, forming a rectangle with sides 150 x 400 feet.

After the wells were sunk into the first salt bed they were double-tubed. The exterior tube is 6 inches in diameter and the interior tube 3 inches.

Fresh water was first let into the interior tube, and after it was saturated it was forced up between the two tubes, as brine. The salt thus dissolved will create a subterranean reservoir, which, as it increases in size, increases in its capacity to saturate the fresh water furnished to the reservoir. Gradually the reservoir at each well will enlarge until, perhaps, in two years they will meet, forming a large reservoir at each group. These will enlarge, until the reservoirs at the different groups unite, and an immense salt basin will be obtained with a rock cover 1 200 feet thick. After a small basin is formed at each well, it is found better to receive the fresh water between the interior and exterior tubes, and deliver the brine through the interior tube.

It now only remained to determine the most efficient method of taking fresh water from a point 23 miles south of the reservoir at Syracuse at an elevation of 820 feet, converting it into brine, and delivering the brine in the most convenient manner for use at Syracuse.

The solution of the problem was very simple.

The necessary dams, gate-house, screen-chambers, etc., were erected at the Tully Lakes to collect the fresh water at the elevation of about 820 feet.

Three miles north, where the surface was at an elevation of about 400 feet, twelve wells were sunk as above described, 1 200 feet into the bed salt. A wooden tank was built at each well, and an open surface reservoir, capacity 1 000 000 gallons, was built about 30 feet lower than the ground at the lowest well.

A 12-inch cast-iron pipe was laid so as to connect the Tully Lakes with the wells, and other cast-iron pipes connected each well through its wooden tank, with the 1 000 000 gallon reservoir.

An automatic gate, regulated by a float in the reservoir, closed the inlet to the 1 000 000 gallon reservoir from the wells when the reservoir was full. When this gate on the inlet pipe at this reservoir closes the brine rises in the wooden tanks. A float in each tank closes a valve on the pipes, delivering fresh water to the wells when the tank is full.

At Syracuse another open surface reservoir of about 5 000 000 gallons capacity was built at an elevation of 112 feet.

The two surface reservoirs, the one at Syracuse and the one at the wells, are connected with 12-inch cast-iron pipes.

The inlet to the Syracuse reservoir is also controlled by an automatic gate, closed by a float when the reservoir is full.

Hence, when the brine is drawn from the 5 000 000 gallon reservoir the surface of the brine therein lowers and the inlet pipe slowly opens. This draft slowly lowers the surface of the brine in the 1 000 000 gallon reservoir at the wells and the inlet gate on the brine pipe from the wells to the 1 000 000 gallon reservoir also slowly opens. This draft lowers the brine in the wooden tanks, and the gates on the pipes which deliver fresh water to the wells, controlled by floats in the tanks, in their turn slowly open.

When the draft at Syracuse is shut off the reverse action takes place.

The whole movement being gradual and automatic, all danger from water hammer is avoided. The necessary gates, air-cocks, blow-offs, etc., along the line are of course provided.

Construction commenced in May, 1889, and was completed in November of the same year.

The plant now yields about 300 000 gallons of brine per day with about 90 per cent. saturation. It is expected that 95 per cent. saturation will eventually be obtained.

The line crosses the Onondaga Reservation, and several of the gangs were composed entirely of Indians. These Indians proved to be the best and most reliable workmen we could have obtained. Their work was most satisfactory when by themselves, with an Indian as a local foreman. It was necessary to treat them with more dignity than ordinary laborers. They would not tolerate abuse. They were not daunted by hard work, heat, nor wet weather. There was but little intemperance among them, and when paid off they deposited the large proportion of their money in the Syracuse Bank, sending it there by a committee from among themselves so as to lose as little time as possible.

I make the above statement in relation to these Onondaga Indians, because the facts differ so radically from the preconceived opinions of myself, and perhaps of others, in relation to this class of people. No doubt our success was largely due to the common sense and unusual tact of the general foreman.

All the work, except the sinking of the well, was performed under a percentage agreement. Notwithstanding the extraordinarily wet season, it was completed within six months, and 10 per cent. below the original estimate.

Annexed to this paper will be found a profile of the main, a section showing the strata at each well, and a sketch showing the principle of the connection at each well.

DISCUSSION.

Mr. Edward N. Trump.—The State Salt Works in Syracuse are not making nearly as much salt as they have made in years past. The output last year was not much over 500 000 bushels, against a maximum output of 1 000 000 bushels six or eight years ago. The falling off in production is not due to any decrease in the strength of the brine, as that has actually increased in strength since we started our works, but it is due to the competition caused by the discovery of beds of rock salt from which could be obtained saturated brine.

With a well 1 200 feet deep the interior 3-inch tube extends to within 6 inches of the bottom of the well and of the bottom of the bed of salt. The exterior tube is only put down far enough into the rock to shut off any veins of water which may have come into the well. If the well were perfectly dry it would not be necessary to put any exterior tube in our wells. We put down an 8-inch tube through any surface soil or sand, until we strike the rock, then inside of this we put a 6-inch tube down about 900 feet, until it reaches some 70 or 80 feet below the vein of water which comes into all the wells at this point. The joint between the tube and the rock is made as tight as possible by driving the tube into the rock and by putting in cement, wheat or beans around the tube. Flaxseed is sometimes used for the same purpose. We get a tight enough joint to stand 900 feet of water around the outside of this 6-inch pipe, so that when the well is baled out from the inside and allowed to stand it is perfectly dry for almost any length of time. If this were not done the water which is forced down the well would travel off into the strata of rock in the same veins in which the water comes, and we should not get brine equal to the amount of water forced into the well.

We have had considerable trouble in the last two months with small leaks on our delivery main. We have found one piece of cracked pipe, but most of the leaks were in the caulking, especially on top of the bell. Many of these leaks it was impossible to see on the surface of the ground, and our attention was only called to them by the condition of the shade trees which happened to be near. Many of them were only the size of a pin, where, perhaps, the brine oozed out and ran down the surface of the metal without any spurting at all. The trees would come out, and after a day or two the leaves would drop down, showing that something was the matter. Our inspector watched the trees along the line, and when he noticed a tree began to show symptoms of failing, by digging down to the roots he would find, sometimes within 20 feet, a very small leak.

We have stopped some fifty or one hundred leaks in this way within the past two months, and seem to have gotten them about all in shape. Some of them may be due to the settling of the pipe.

These leaks show that it is probable that water mains leak large quantities of water without it being noticed at all on the surface of the ground. Some of the leaks ran quite a large stream without showing.

The wells are all running nicely, and since changing the method of operation to forcing around the outside and up the center tube the brine has become stronger, and we now get the full supply for our works at 95 per cent. saturation. As soon as we get the benefit of our new wells we hope to supply the salt-makers.

John Bogart, M. Am. Soc. C. E.—Is Onondaga Lake brackish?

Mr. Brush.—Yes, quite brackish. At Station 8 a well was sunk, as shown on Plate XVII, but no salt was found. Further investigations developed the fact that the salt beds lie between Stations 9½ and 10½, about 10 000 feet in width. Beyond these stations slate rock was encountered under the limestone rock, instead of salt.

A Member.—What is the depth of the wells at Syracuse?

Mr. Brush.—I understand about 300 feet. Mr. Trotter can probably inform us as to the depth of the wells at Piffard and Warsaw.

A. W. Trotter, Jun. Am. Soc. C. E.—The geological section is probably about the same from Syracuse to Buffalo. In the Warsaw Valley the depth is about 1 600 to 1 800 feet to salt.

Mr. Brush.—My diagram shows that the sea level is 772 feet below the surface of the ground, and that the salt is struck 300 feet below sea level.

Mr. Trotter.—That fact may prove true all the way to Buffalo—that is, for that elevation of the ground. It is about the same as in the Genesee Valley where salt is found. But if the same general fact is true, as Mr. Brush has stated, that salt is not found much north of Station 9½ on the diagram, the fact of only finding salt below sea level would also, I think, prove true elsewhere. Of course, the further south you go the further below the sea level the salt would go, following the dip. The fact of its being found 300 feet below sea level there and at Piffard, is probably only true because the elevation of the surface of the country is 700 feet above, so far as we may judge from our present insufficient data.

T. H. McCann, M. Am. Soc. C. E.—Was the lining of the pipe the ordinary lining, and was there any effect from the action of the salt?

Mr. Brush.—The lining is ordinary iron tubing, and it will have to be renewed from time to time.

The annual report of the Superintendent of the Onondaga Salt Springs, for the year 1889, makes this statement in reference to the brine obtained from these wells :

"For the profile of the brine pipe line accompanying this report I am indebted to the kindness of the officers of the Solvay Process Company. It requires no explanation. There are two facts, however, connected therewith to which I wish to call your special attention, since, in my opinion, they are of the utmost importance, namely: First, the salt is found directly under the limestone; therefore, no caving in of these wells can occur, which gives so much trouble, and is so expensive, both in time and money, to the manufacturers in the western part of the State; and, secondly, by taking the water from the Tully Lakes, situated 300 feet above these wells, it saves all the expense for pumping, since the pressure exerted by this fresh water through the pipe forces the brine to the surface, and from there it comes, by gravity, to Syracuse. Ten more wells have now been contracted for by the Company, to be sunk at once. With these wells in operation, sufficient brine can be obtained to make 4 000 000 bushels of salt annually. In regard to the quality of the brine now obtained, I submit the following analysis, which I have made of a sample of brine obtained from the Company: Specific gravity, 1.1861, at 60 degrees Fahr. (water and brine at same temperature):

Sulphate of lime	0.5862
Chloride of calcium	0.0123
Chloride of magnesium	.0115
Pure salt	23.6530
Water	75.7370
	100.0000

"These results compare most favorably with the analyses of the best brines of this country."

PLATE XVII.
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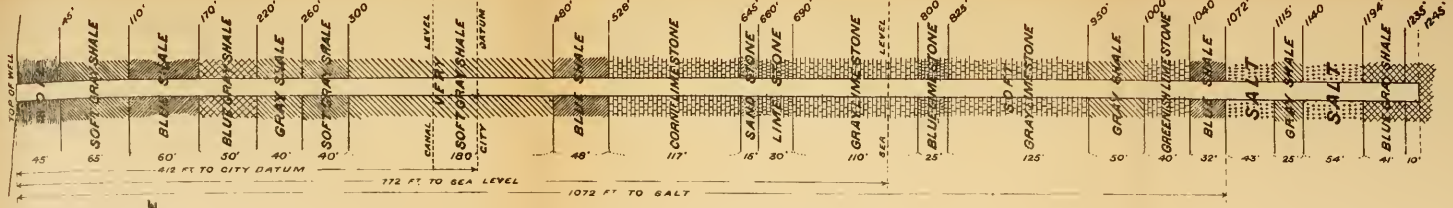
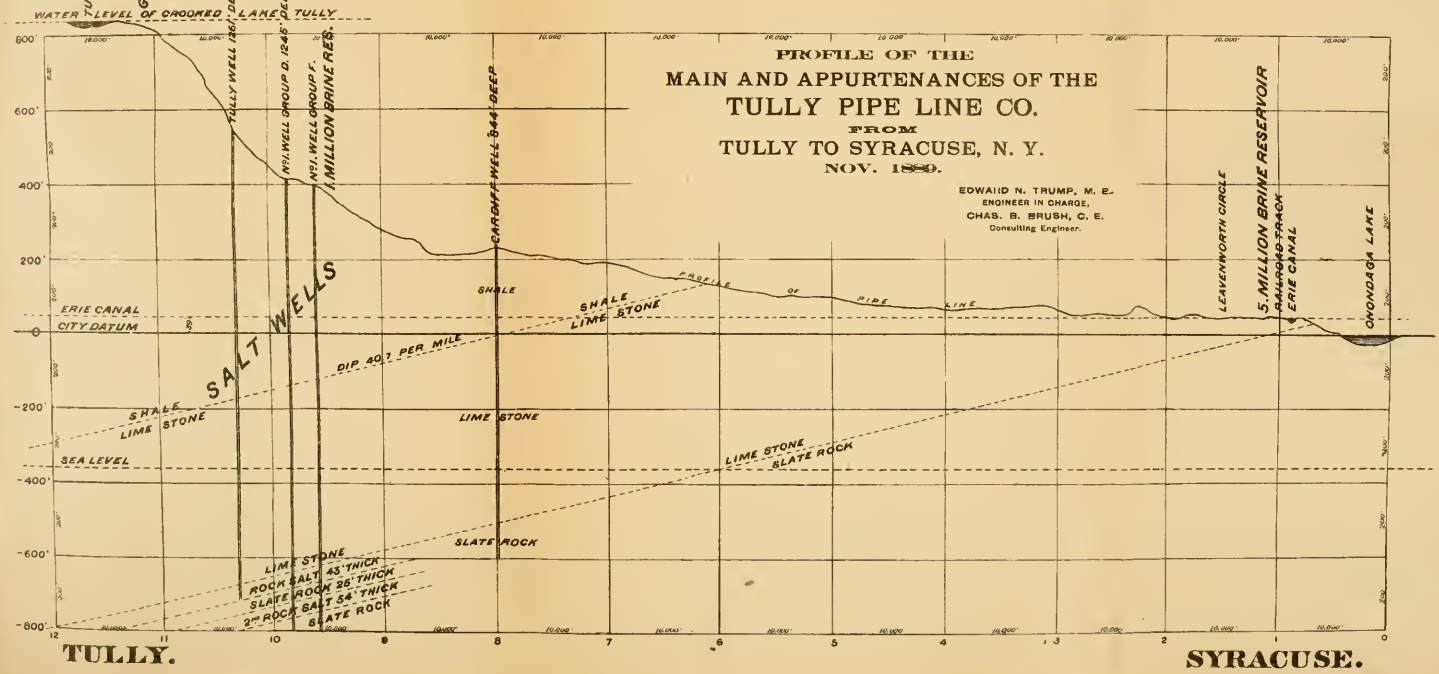
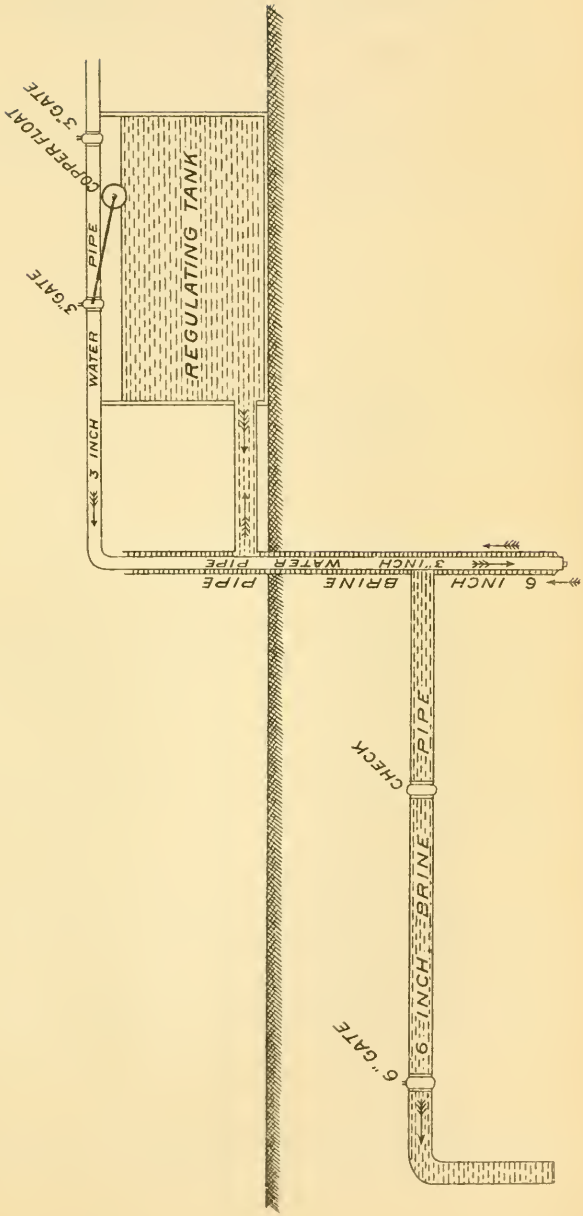
BRUSH ON
OBTAINING BRINE



SECTION OF STRATA
At TULLY BRINE WELLS,
TULLY, N. Y.





SKETCH SHOWING THE PRINCIPLE OF THE CONNECTION AT EACH WELL.

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Vol. XXIII.—August, 1890.

ON THE CAUSE OF TRADE-WINDS.

By FRANZ A. VELSCHOW, C.E.

WITH DISCUSSION.

Maury's theory on trade-winds which, up to the present day, has not been superseded by any other, though admitted to be highly unsatisfactory and even contradictory, was formed at a time when the materials on hand of actual observations were very scanty. At the beginning of this year I formed another theory which to me seems to meet the case, and when I, not long ago, crossed the Atlantic in the teeth of the anti-trade-winds, the subject naturally recurred to my mind, and so I shall here try to set forth the opinion I have arrived at, as it may cause other engineers to express their valuable opinions on the subject.

The theory now in existence was founded mainly on the following observations: 1. Ehrenberg's observation that certain animalcules found by him in the sand dust on the islands of Cape Verde were supposed to have been carried there by the wind from Brazil, as he knew of no other place where such animalcules were to be found; and 2. The rotary motion of the earth.

1. As to Ehrenberg's observation already the late Admiral Fitzroy says in his "Weatherbook:" "Similar animalcules were afterwards

found in sand dust on islands in the Mediterranean Sea, and these were also supposed to have been blown from Brazil, though the Sahara lies between." It is well known that heaps of sand are carried from the Sahara to the islands of Cape Verde by the violent gales "the Harmattan," and also into the Mediterranean Sea by the Zirocco or Foehn-wind (Föhn), as it is called in Switzerland, and it is therefore highly probable that Ehrenberg's animalcules never came from Brazil but from the Desert of Sahara. However, Ehrenberg's observations, says Admiral Fitzroy, furnished the celebrated Captain Maury with two fixed points whereupon to start his theory on trade-winds. As the surface wind between these two places is nearly constantly blowing from the islands of Cape Verde towards Brazil the animalcules were supposed to have been carried by an upper or counter-current from Brazil to the islands of Cape Verde. To re-establish equilibrium the existence of counter-currents appeared a very natural supposition, particularly as, at that time, the cycling movement of the surface air had not as yet been pointed out. The whole question of counter-currents at a higher level was hereby brought conspicuously to the front; but it seems now that they can only have a very slight or secondary effect upon the atmospheric phenomena at the earth's surface, as there exists a decided relation between the direction of surface winds and the general distribution of pressure. Upper-currents, therefore, do not appear at all in our explanation of the trade-winds.

2. As to the earth's rotation: railroad engineers will have observed that trains running north and south press with their wheels harder against one rail than against the other when running one way, and against the other rail when running in the opposite direction, and that this is owing to the earth's rotation. But for meteorologists to conclude therefrom that a body of air moving north and south over the earth's surface should change its direction for this same reason seems hardly to be admissible. A current of surface-air is, so to speak, moving in a tunnel on the bottom of the ocean of air, being on both sides and above closed in by other air which, by being in a state of relative rest has, by itself, a rotary motion corresponding to the latitude on which it is found. To be able to curve, the current of air would have to shift this surrounding air, which seems very improbable. But we see clearly that the argument must be wrong, because the current of air curves not only when the wind is of a north and south direction,

but exactly in the same manner and degree when (take for example the case of the current round an anti-cyclone) the wind is blowing east and west. In this latter case the curvature cannot be owing to the earth's rotation; but if it is not the cause then, neither can it be when the wind is blowing north and south, as these exactly similar phenomena must have one and the same cause.

This rotation theory is, however, supported by the coincident that the surface air rotates in opposite directions in the two hemispheres, and it appears as a disturbing element in a great many meteorological explanations. It is, for instance, set forth as a general maxim that to establish equilibrium the air in a high-pressure will tend to move outwards from the center in all directions, and inwards toward the center in a low pressure, but, owing to the earth's rotation, the wind curves off from its original direction. The result is that in an anti-cyclone we find a current of surface air moving round the border of it. If this current, however, was really a manifestation of a tendency to establish equilibrium, we should find the pressure gradually lowering at the center ; but strangely it appears that certain anti-cyclones remain more or less permanent in spite of this current ; and not only they remain permanent nearly all the year round, but it appears that when the border current is at its strongest, that is just the time when the pressure is increasing at the center (we are here thinking of the North-Atlantic anti-cyclone). One would naturally conclude from this that the current is somehow carrying air toward the center rather than the opposite. Of the motion of air round a low pressure or cyclone we find a similar explanation. There is no cause given for the origin of the low-pressure or vacuum, but it once being there the air, while moving towards it from all directions in trying to fill it out, bends off from its original course owing to the effect of the earth's rotary motion, and so, having once missed the center, keeps on circling round it, always trying to reach the center, but never appearing to get there.

It will not be necessary to enter upon the complicated theory which has been formed on these two observations, but it may here be observed that it leads to the monstrosity that the trade-winds, after having blown across the earth's surface, are supposed to rise to a higher level, there to pass through each other and then come down again to the surface as anti-trades (having changed their direction in the meantime owing to the earth's rotation), as if it was possible to imagine that the wind could blow at any point in two opposite directions at the same time.

The earlier observations on the direction of the trade-winds have mostly been made by seafaring people, and as it was observed that the direction of the winds over the adjoining land surfaces does not follow the same rules as could be applied at sea, so it was concluded that the trade-winds were something exclusively pertaining to the sea surface, and the theory we have mentioned was formed accordingly, entirely neglecting the winds from the land.

If we now would try to uphold a different theory on trade-winds we may commence by observing that the principal cause of the aërial movements we are here to discuss appears to be the varying degree of surface evaporation at different localities. The heat from the sun is generally recognized as being the bottom cause of all aërial disturbances, but if there was no surface evaporation, or a perfectly uniform surface evaporation, the sun's heat would be communicated to the earth in such a uniform manner that one can hardly imagine a cause for any wind at all. Where an ordinary degree of surface evaporation takes place, about half the sun's heat is utilized in transforming water into invisible vapor, the other half in heating the surface and surface air. Where no evaporation takes place, as in deserts and dried-up places, all the sun's heat goes to heat the surface and the surface air.

I must here beg to refer the reader to my pamphlet on the cause of deserts and droughts,* in which it is maintained that the effect of absence of surface evaporation is the development over arid districts of a layer of highly unsaturated surface air, or, what I call an air cushion, which makes it impossible for rain to come down on the dried up land, as long as it prevails over it. I also suggested there that the dry and heavy surface air would be likely to run off the land, and we shall now see that it does run off the land, and for some reason or another always in a westerly direction (that is in a direction opposite the way the earth rotates) out over the oceans adjoining the deserts. It is the passage of this dry air over the sea surface which we call the trade-winds.

The subjoined map of the world, Plate XIX, is taken from R. H. Scott's Elementary Meteorology, 1887. It shows the mean pressure and prevailing winds for one calendar month. But I have here taken the map of the Northern Hemisphere for July, together with the map of the Southern Hemisphere for January, as to be able to find out the analogous

* The Natural Law of Relation between Rainfall and Vegetable Life. Stanford, London, 1888.

atmospheric features of the two hemispheres we must compare the two hemispheres when they are in the same intermediate position with regard to the sun. The most prominent feature to be observed on this map is a stationary high-pressure or anti-cyclone prevailing over each of the five great expanses of oceans and the wind circulating round them according to Buy Ballot's law; that is, turning to the right in the Northern Hemisphere and to the left side in the Southern.

The persistency of these high-pressures is indeed remarkable when it is remembered that high-pressure means dry air and clear sky. How then does it happen that this dry air exists over the oceans, and how can this dry air keep itself up against the lively evaporation from the sea surface when the sun is always shining on it?

On Plate XX, I have inserted the rainless districts of the earth, and it will be noticed that to the east of each high-pressure, a little nearer the equator is found a desert or a drought district (which latter are perfectly dried up and therefore practically deserts during summer). Over each of these dry districts dry surface air is constantly developing and accumulating; surely therefore there must be a connection between these dry localities and the dry air accumulated to the west of each of them over the sea surface.

I shall now point out the connection between the Sahara and the great, almost permanent, anticyclone in the North-Atlantic Ocean at Horse-latitudes, the explanation of four other instances of high pressures mentioned being perfectly analogous.

On Mr. Scott's map is not given any indication of the prevailing winds over the Sahara, but it is well known that the wind blows constantly from the east over the Sahara during nine months of the year, carrying enormous masses of sand from the desert into the Atlantic Ocean, and the violent Harmattan gales on the west coast of Africa are felt hundreds of miles out at sea, where the ship's decks often get covered by sand dust from the desert.

Let us now examine what becomes of this hot and dry air which is constantly pouring out from the desert over the North-Atlantic, and let us try to explain the trade-winds by this simple fact. Taking before us the time of the year when the wind for a few months has ceased to blow from the desert, we find that the Horse-latitude anti-cyclone over the Sargasso Sea is generally broken up and the pressure over this part of the North-Atlantic more or less indifferent. When then a Harmattan

gale bursts from the Sahara the body of hot and dry surface air which thereby is brought out over the ocean, will, for obvious reasons, cool down. It gets thereby condensed, and the air, which in its expanded state when over the desert showed no remarkable high pressure, will in its more condensed state over the sea surface soon appear on weather charts as a center of high-pressure. Howsoever the high-pressure commences, the wind which is constantly blowing from the desert must bend off from this high-pressure either to one side or the other; but it so happens that the high-pressure always locates itself a little to the north, and consequently the wind which comes straight east and west from the Sahara is bent off in a southwesterly direction along the west coast of Africa. The dry air of this current, however, appears to adhere to the body of dry air already found over the ocean, and so the current from the desert follows the high-pressures all round its circumference. The initiating cushion or "island" of dry air resting over the ocean is thereby gradually increasing in bulk as the circulating current is constantly adding fresh supply of dry air to it, or in other words, this current acts as a feeder of the anti-cyclone. Without this feeding process the anti-cyclone would soon die away, owing to the evaporation from the sea surface, which, by making the air lighter, acts against the high-pressure.

We may simply describe the whole proceeding by saying that the desert air is running into a pool over the sea surface, as the reader will have observed that whenever we cause a fluid to run into a pool, say for instance water running into a cavity on the floor, the inflowing fluid does not run in straight, but is always circulating round the center of the pool, thereby gradually increasing it in bulk.

Plate XIX shows the anti-cyclone fully developed. The air on both sides of the feeding current is drawn by it in a direction towards it, and we thus see the complete system of trade and anti trade-winds plainly before us.

Plate XX, which follows the directions of the prevailing winds, as shown on Plate XIX, as closely as conformable to my way of reasoning, merely shows the main or feeding current of the anti-cyclone.

From this explanation of the cause of trade-winds may be drawn some conclusions not without a certain general interest. In the first place, it will be noticed that the existence of this permanent North-Atlantic anti-cyclone depends directly upon co-existence of the Desert

of Sahara, and it is well known how much the state of weather in these latitudes is dependent on this anti-cyclone, as cyclone after cyclone follows each other round its borders. It was suggested some years back to try to submerge the Sahara, and it was also surmised then that such an undertaking, if it could be carried out, would be likely to cause an entire change in the climate of Europe. We may now perhaps better realize what would be the effect on the climate of Europe, and America too, for that matter, if the desert was put under water, and it seems hardly to be a desirable one.

This anti-cyclone or air cushion is always moving slowly in one direction or another, and when sometimes it reaches in over the west coast of North America it gives warm weather to these shores, and the wind there is then always blowing from the south. When, however, people in this country would argue that the weather is warm then because the wind comes from the south, it may now appear that the weather is warm not because the wind is from the south, but because this air has come across the Atlantic from the Sahara, and then turned into a northerly direction. It takes, namely, a very considerable time for a large body of heated surface-air to get cooled down, and the circulating current of desert air is still very warm when it has crossed the Atlantic once more on the north side of the center of high-pressure and approaches Europe. It causes some perplexity to meteorologists in England to explain why the warm weather then generally comes from a northerly direction; as will appear from the map, the current from the desert here takes that direction round the high-pressure center. It thus sometimes happens that late in autumn a period of warm and fine summer weather suddenly sets in, after a long continuation of cold and rainy weather; but in all such instances hereof, which I have had before me, Western Europe has been covered by the North-Atlantic anti-cyclone, so here we meet the heated air from the Sahara once more.

It may be as well to mention here that the difference in temperature between night and day does not appertain to the atmosphere as a whole, but is a mere surface effect, owing to the radiation of heat into space. Thus the temperature in the desert may on a clear night sink as low as zero after the hottest day, but if we were to go up in a balloon a hundred feet or so, we would find the air there as warm as during the daytime; and also if we dig down a foot or so in the sand we would find it equally hot there. When the sky is overcast so as to prevent radiation,

the air in the desert hardly changes its temperature from day to night, and I have experienced in the interior of Australia 112 degrees Fahr. during a clouded night.

The trade-winds of the North-Atlantic are stronger or more pronounced than those of any other ocean. But the desert area over which the dry air originates is also the most extensive, as it consists not only of the Desert of Sahara, but further to the east we find the rainless district of Arabia, Persia and the great Desert of Gobi in Central Asia. Over all this long tract of dried-up land the so-called perennial winds are constantly moving the surface air from east to west till they finally get consumed by the North-Atlantic anti-cyclone.

If we look at Map II once more it will be noticed that the five trade-wind anti-cyclones are all located a little further from the equator than the dry tracts of land from which the dry air pours out over the oceans. The reason hereof I shall not attempt to explain, but the consequence of this fact is that the circulation of wind round the centers of high pressure takes place in opposite directions in the two hemispheres, according to Buy Ballot's law. It cannot be maintained, however, that this circulation in opposite directions is in any way owing to the earth's rotation, *e. g.*, the east winds from the Sahara are thrown into a southwesterly direction by the North-Atlantic center of high pressure, and that is to bend in a direction contrary to Buy Ballot's law.

The general conclusions which may be deduced from this explanation of the trade-winds are as follows: (1.) The surface air becomes dry by resting over the large arid zone lying north and south of the equator, and is formed into cushions of dry air which are ultimately thrown westward over the surface of the oceans. (2.) By getting cooled down over the sea surface the dry air appears there as centers of high pressure which are kept up or fed by the continual flow of dry air from land which circulates round these centers of high pressure. (3.) Each trade-wind, therefore, in each of the five oceans is one continual current of dry surface air issuing from a dried-up land district lying to the east of them. (4.) The direction of the circulation does not depend upon the earth's rotary motion, but upon the interposition of the centers of high-pressure and the corresponding dried-up land districts, the former always lying a little further from the equator than the latter.

As the dry air which causes the whole trade-wind motion is produced

over the dry land surface and consumed or saturated by evaporation in the centers of high-pressure over the oceans, it will be observed that there is a certain natural sequence in the explanation given, and it enables us to trace the trade-winds from their place of origin to where they ultimately die out.

DISCUSSION.

Charles Macdonald, M. Am. Soc. C. E.—The action of the Atlantic Trades, as indicated in the paper just read, should be of special interest as offering a very probable explanation of the existence of the Gulf Stream.

I believe that the old idea of the discharge of the Gulf Stream from the Gulf of Mexico has long since been abandoned; the records of the Coast Survey having shown that the water space between Florida and the island of Cuba is utterly inadequate to pass the volume of water known to exist in the Gulf Stream.

It would seem reasonable, therefore, to assume that the steady pressure of warm winds from the coast of Africa toward the west and north should force tropical waters over toward the North American coast, along which the current would naturally take a northeasterly direction.

There is a current setting easterly through the Indian Ocean along the Equator, from the coast of Africa, which, in all probability, is the result of a similar action of the winds to that described in the paper; the southeast Trades forcing the water against the east coast of Africa, which deflects it back to the eastward in a stream which has a velocity of about sixty knots per diem. A small fraction of this stream finds its way north between the island of Socotra and Cape Guardafui, causing, at certain periods of the year, dangerous "cross-seas."

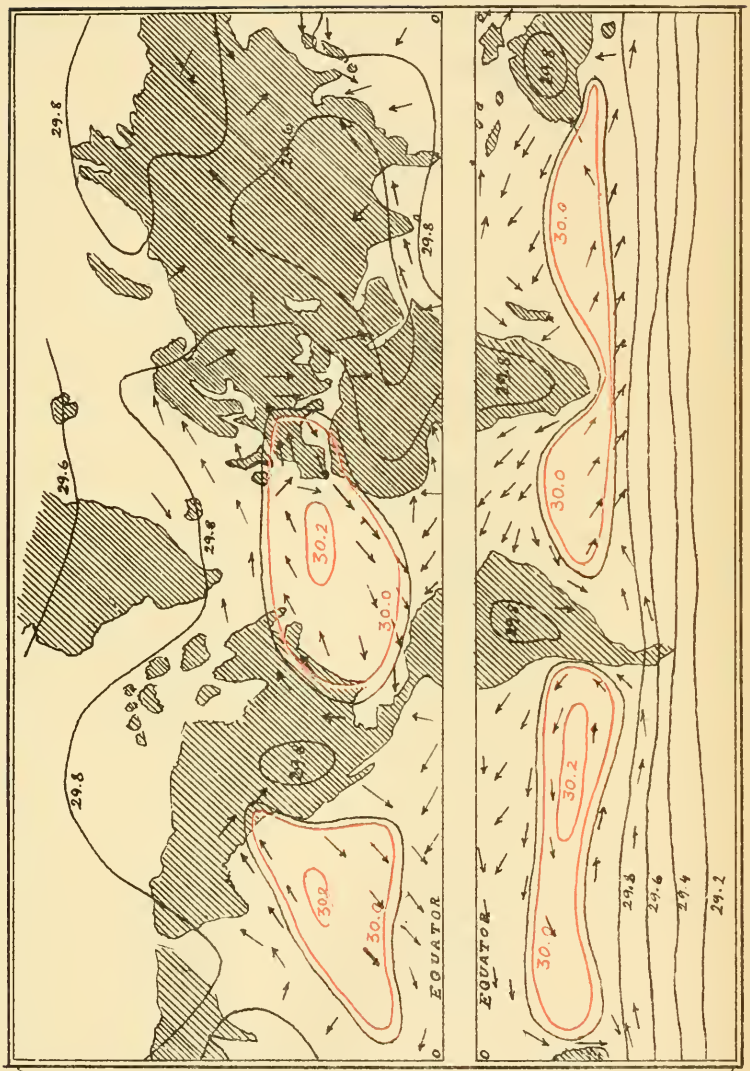
Mr. Velschow's diagram is well worth the most careful study.

The plates accompanying the paper by Mr. Velschow on the Cause of Trade-Winds are:

Plate XIX.—Map showing Mean Pressure and Prevailing Winds.
" XX.—Map showing Rainless Districts and Trade-Winds, and their Relation to Stationary High Pressure.

PLATE XIX
TRANS. AM SOC. CIV ENGRS.
VOL XXIII. Nº 448.
VELSCHOW ON
CAUSE OF TRADE WINDS.

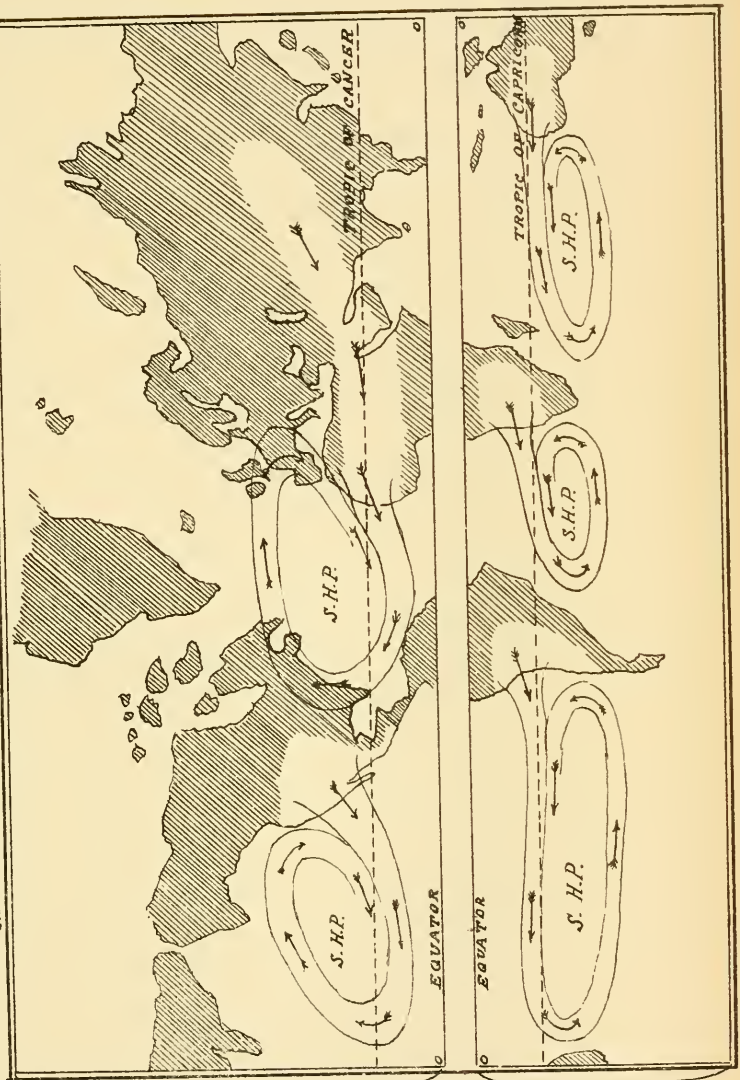
R.H.SCOTT: MEAN PRESSURE AND PREVAILING WINDS.



FOR MONTH OF JULY

FOR MONTH OF JANUARY.

F.A. VELSCHOW: RAINLESS DISTRICTS AND TRADE WINDS
AND THEIR RELATION TO STATIONARY HIGH PRESSURES.



FOR MONTH OF JULY

FOR MONTH OF JANUARY.

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449.

(Vol. XXIII.—September, 1890.)

THE CHEAPEST RAILROAD IN THE WORLD.

By ARTHUR PEW,* M. Am. Soc. C. E.

WITH DISCUSSION.

In the construction of most railroads in the old world, and also in the older and richer sections of the new, the problem usually solved by the engineer is, simply how to effect the most judicious expenditure of the company's money and build economically the road best adapted to the traffic that is to sustain it.

If from a careful estimate of the probable business of the road, and of the cost of operation, it appears that light gradients, long tangents, iron trestles, stone ballast, a heavy rail, etc., would be profitable investments, they are usually adopted; but in poor or sparsely settled localities, especially in some of the Southern and Western States, it is often very difficult to procure enough money to build any kind of a railroad whatever, and the engineer is often called on to make surveys and estimates for the very cheapest and poorest railroad his conscience will allow him to build. Then, the problem to be solved is, "Given a very small amount of money, required any kind of a railroad that will serve the immediate wants of the section and encourage settlers to move in and develop it."

* Formerly spelled Pou.

A few years ago, it was the fortune of the writer to assist the president of a railroad in the solution of a noteworthy problem of this character. It resulted in the construction of what is supposed to be the cheapest railroad that has ever been built to do a regular freight and passenger business.

Its history is as follows: In the fall of 1884 it was decided to make an effort to build a railroad from Wrightsville to Dublin, two Georgia villages about 19 miles apart.

The prime movers of the enterprise, after canvassing the line of the road and the country generally, called a meeting at Dublin, organized under the name of the Dublin and Wrightsville Railroad Company, obtained a charter, and secured subscriptions to the Capital Stock to the amount of $22 600. Mr. W. B. Thomas, of Tennille, Ga., was elected President and Superintendent, and his judicious management was the principal factor in the success of the road.

It was estimated that three-fourths of the amount subscribed could be collected in instalments of 10 per cent. per month.

The remainder was considered as worthless, but it was hoped that a sufficient amount of new subscriptions could be obtained to make up the deficiency.

It was now assumed, that, if the road-bed could be made ready for the rails with the amount subscribed, rails and rolling stock could be procured on easy terms from the Central Railroad of Georgia (of which it would be a feeder), or, that the road could be bonded for a sufficient amount to purchase them, so it was decided to build the road, and the writer was appointed Chief Engineer.

The country through which the line passes is moderately rough.

The Ohoopee River and Bruton Creek, both nearly 100 feet below the general level of the country, are crossed, besides several smaller streams, and an air line would have given very heavy work. On the route selected, however, the hill-sides were long and smooth, and the line could be fitted to them with easy curves and light work.

The location was soon made, and grading commenced with about forty convicts. The greater part of the line was through pine forests, and at first the clearing was only done for the width required by the grading. After the grading was completed several miles ahead, the convicts would return and clear the full width of the right of way, all suitable timber being hewn into cross-ties, and the remainder being preserved for fuel for the engines.

The price paid for the convicts was a dollar a day each. The grading done by them cost about nine cents a cubic yard, and the cross-ties nine or ten cents each.

As the grading was very light, the roadway was made 14 feet wide in cuts and 10 feet wide on fills, the intention being to widen both as much as necessary out of the earnings of the road.

With this narrow roadway, the grading averaged something less than 4 000 cubic yards per mile.

Convicts were also employed, under the direction of skilled foremen, in hewing bridge timbers and building trestles and, owing to the abundance of first-class timber, many trestles were built entirely of trees that had grown within 200 feet of the structure. These trees were cut down, hewn to the proper dimensions by expert axmen, framed, and put into trestles at less than half the usual contract price.

The President's first annual report to the stockholders, shows that on December 1st, 1885, the line had been located, nearly all of the right of way secured by donation, and that 11½ miles of the road had been completed and was in operation.

The track was of standard gauge, was laid with a 45-pound iron rail, fastened with fish plates, full-bolted, and of course the rails were full-spiked.

The ties were of first-class Georgia pine, hewn on two sides, and were laid at the rate of 2 640 to the mile.

The total cost of preparing the road-bed for the rails had been as follows:

Right of way purchased	$700 00
Engineering Department*	1 134 19
Grading, trestles and cross-ties	11 557 45
Total	$13 391 64

Excluding the items of engineering and right of way, the 11½ miles completed had cost $1 005 per mile for clearing, grubbing, grading, trestles, cross-ties and ditching, or $1 164.49 per mile with engineering and right of way included.

The cost of a railroad, depends entirely on the various conditions under which it is built. In this instance, all of the conditions were

* The small cost of engineering was due to the fact that the engineer only devoted a small part of his time to the road after the location was completed.

specially favorable, and as stated, the road-bed was made ready for the rails at a cost of $1 164.49 per mile.

Where all the conditions are specially unfavorable, a road may well cost forty to fifty times as much.

There is an instance on record where the simple survey of a railroad cost £445 per mile. The country was very rough and unhealthy, and the engineers labored under great difficulties, but estimating the £ at $4.90, the actual cost of the survey alone was $2 180.50 per mile.

The track on the Dublin and Wrightsville Railroad was also unusually cheap. A good iron rail, varying in weight from 40 to 45 pounds per yard, was furnished, and laid for $1 800 per mile.

This could not have been done had not the company been favored by a fortunate circumstance.

The Central Railroad of Georgia, just at this time, had on hand a large quantity of iron rails, in good condition, which had been taken up from one of their branches to be replaced by a heavy steel rail. As the Dublin and Wrightsville would be a valuable feeder for it, the two companies entered into a contract, of which the President of the Dublin and Wrightsville, in his report, says :

"This contract was in substance as follows : That that Company would furnish good second-hand rails, frogs, switches, joints and spikes, and lay and put the track in good surface, from Wrightsville to Dublin, for $35 000, payable in preferred stock at par value, with 6 per cent. preference, out of the net earnings of the company, * * * * and that this company should be free from debt, and should finish grading, bridging, etc., in a good and substantial manner, ahead of the work, and furnish the ties in easy and convenient reach of their construction train."

The distance was estimated at 19½ miles, making the cost of the work done by the Central Railroad approximately $1 800 per mile.

This was probably less than its real value, but that company generously did their part very cheaply, in consideration of the economical manner in which the other work had been done.

Dublin, the objective terminus of the road, is on the west side of the Oconee River (a navigable stream), and as it would have cost $15 000 to $20 000 to have bridged the river, the company was forced to be content with touching the corporate limits of the town on the east bank.

It was at first proposed to stop near high water mark, but the road was finally built to a bluff on the bank of the river; a wharf was constructed, and a track run to it for convenience in receiving and shipping freight by steamboat.

The extension of the road from the high table land down to the bank of the river was very expensive; the last mile and a half contained a long trestle and a long deep cut, and, excluding the track, cost more than five times as much per mile as the rest of the road.

The second annual report of the President, shows that the road had been completed to the bank of the river, the distance being 19 miles.

The total cost for engineering, right of way, salaries of officers, grading, cross-ties, cattle-guards, road-crossings, depot buildings, water-tanks, wharf, etc., was $30 380.50, or $1 598.97 per mile; adding $35 000.00 for track, sidings, etc., gives $65 380.50 as the total cost, or $3 441.08 per mile for the road ready for rolling stock.

The character of the business done, was such that nearly all of the freight was received and shipped in foreign cars, so that $1 000 a mile would have been ample to have secured all of the rolling stock needed, and the cost of the road and equipment complete (had this been purchased), would have been $4 441 per mile.

It was not necessary to buy rolling stock, however, for the following reason :

The Dublin and Wrightsville, is simply an extension of the Wrightsville and Tennille Railroad. This road had been constructed from Tennille (a station on the Central Railroad) to Wrightsville, a year or two before, and as the company had purchased all the rolling stock necessary for its immediate wants, and as the combined length of the two roads was only 35 miles, a contract was made by which the Wrightsville and Tennille trains were to run on both roads, and the expenses of running them were to be divided.

This arrangement resulted in a considerable saving in the train expenses of both roads, and it was of special advantage to the Dublin and Wrightsville, as it obviated the necessity of purchasing rolling stock, so that the actual cost of the road in operation with chartered rolling stock was $3 441.08 per mile.

The following notes in regard to the rise and fall, and curvature, will give some idea as to the character of the country and of the road.

About a mile and a quarter after leaving Wrightsville, the road crosses

a ridge through a gap 20 feet higher than the starting point. Within the next mile and a half, it descends 76 feet to the valley of the Ohoopee River. Two miles further it has left the valley of the Ohoopee, has ascended 87 feet, and is on a smooth ridge. This is followed a mile, ascending 11 feet, when the summit of the road is reached. From this summit, descending gradually for 3 miles, and rapidly for 1 mile, it crosses Bruton Creek at an elevation 138 feet below the summit. Following the valley of this creek a mile and a half, a small tributary is crossed at the same elevation, and is followed a mile and a quarter to its source, and the ridge is crossed 88 feet above the grade line at the bottom. Within the next mile and a half, it descends 73 feet, and is then on comparatively level ground till the descent to the Oconee River is commenced. The depot is in the valley of the Oconee, 186 feet below the summit.

These notes will show that the line was certainly not on a level plain, and to locate it so that the grading should be very light required a considerable amount of curvature; but great care was exercised to use as little as the finances of the company would admit of, and to arrange that used in the least objectionable manner.

In the 17.6 miles located by the writer, there are 10.8 miles of tangent, and 6.8 miles of curves. Of this distance, 6 miles are of curves varying from 30 minutes to 4 degrees deflection per station, and 0.8 of a mile is of 4½, 5 and 6 degree curves. The 6 degree curves are the sharpest used.

The other mile and a half was located by another engineer and the notes have not been obtained as yet.

There are no reverse curves on the road, and the shortest distance between any two curves is 140 feet.

The shortest curve is 353 feet in length, with a central angle of 7 degrees and 0.4 minutes. The longest curve is 4 000 feet, with a central angle of 71 degrees and 30 minutes, which is the largest central angle on the road.

The average length of curves is 1 178 feet, and the average central angle 37 degrees 20 minutes. The ruling grade is one and a half per cent. on tangents, with compensation for curvature.

The engineer was instructed by the Board of Directors to locate the line by the village of Condor. The air-line distance from Wrightsville to Condor, plus an air-line from that point to Dublin, is 5 200 feet

shorter than the distance from Wrightsville to Dublin by the line of the road.

Of course a better line could have been secured, by the expenditure of more money, but the company had very little money, and could not expect it to build a first-class road.

It has been suggested, that it would be of interest, to have an estimate of what would have been the cost of the road, had it been laid with new steel rails, but the writer prefers not to insert such an estimate, for the reason that it would lead to estimates as to the cost of other changes in the method of construction, and these could be extended indefinitely.

The object of this paper is not to discuss the cost of railroads, nor to teach any new principles in railroad construction, nor to advocate the adoption of the methods described, but simply to describe a railroad as it is, explain how it was built so cheaply, and place it on record as claiming to be the cheapest railroad in the world. Toy railroads, or those built for logging or other temporary purposes, have doubtless been built for less money, but it is intended that this shall be compared only with permanent roads, doing a regular freight and passenger business.

American railroad engineers seldom give to the public, the exact cost of the work under their charge, and this paper would not have been written, had not the principal facts and figures it contains, already been given to the public, through the published reports of the President and "Poor's Manual."

The subsequent history of the Dublin and Wrightsville road is, that soon after completion, it was consolidated with the Wrightsville and Tennille Railroad, which, though several miles shorter, had cost more, was laid with very inferior rails, and contained some objectionable grades and curves.

The two roads at present form the Wrightsville and Tennille Railroad.

Its length is 35¼ miles.

Capital stock	$119 622 21
Bonded debt	35 000 00
Total stock and bonds	$154 622 21

Since the completion of the road to Dublin, the earnings have always exceeded the operating expenses, and from this surplus, and by assuming a bonded debt of $35 000, the roadway has been widened, objectionable grades and curves have been improved, new depots have been erected, additional cars and engines have been purchased, 20 miles of new steel rails have been bought and laid, and an annual dividend of $6\frac{2}{3}$ per cent. has been paid to the stockholders.

DISCUSSION.

A. M. Wellington, M. Am. Soc. C. E.—I heard all about that railroad at the time it was building, Mr. Pew having been in considerable doubt whether it should not be built to narrow gauge. I advised strongly against this, and it now appears, with good reason, on the ground that a broad gauge road could be built just as cheaply if desired. The narrow gauge movement is so far in the past now that it is hardly worth discussing that phase of the question, as it is now generally admitted that the gauge has very little to do with the cost of a road; but Mr. Pew's paper would have been a very opportune one a decade or so ago.

D. J. Whittemore, Past President Am. Soc. C. E.—Facts sometimes indicate how some cheap railways are built. Many years ago a bankrupt company was forced to part with its road and applied to the president of the corporation with which I was then connected as engineer, to become its purchaser. The line was most wretchedly built, but as it connected two important points 36 miles apart it was thought wise to secure it. The presidents of both companies were canny Scotchmen, and of course both were intent on securing the best bargain they could. During the conversation that led to the purchase, the purchasing president said to the seller: "We think from what you represent as the cost of this line that you did not exercise due economy in its building." The answer was promptly given that, "The utmost economy was exercised. Why, we paid only $500 for engineering for the whole line of 36 miles."

The line was purchased and it became my duty to rebuild it from the grass roots up, and during this process it was a constant puzzle to me, and I venture to say that it would have been to the most acute member here present, to determine just where the $500 had been spent for engineering.

Arthur Pew, M. Am. Soc. C. E.—The Dublin and Wrightsville Railroad being an extension of the W. and T. R. R., and having no

connection with any other road, was naturally designed to have the same gauge.

Mr. Wellington is mistaken in supposing that it was ever proposed to make it a narrow gauge.

During its construction I was also Chief Engineer of the Macon and Dublin Railroad, then being discussed, and which the Directors at first proposed to build as a narrow gauge. I protested against this and had some correspondence with Mr. Wellington on the subject, and he, knowing that I was connected with the Dublin and Wrightsville Railroad, naturally supposed that to be the road referred to.

The fact of my being in charge of both roads caused the engineering expenses of the Dublin and Wrightsville to be considerably less than they would otherwise have been, as after the line was located that company only paid for the time necessary for cross-sectioning, etc. But all the engineering work was done as carefully as if the road was to be a costly one instead of a cheap one.

The fact that a large amount of curvature was necessary, was not supposed to warrant its use on all occasions. Care was exercised to avoid specially objectionable forms of it, such as reverse or short "broken-back" curves, and transitions were used with all 5 and 6 degree curves.

The company did not make the mistake of trying to economize in the engineering, and it was generally considered that the care with which the location was made, was an important element in assuring the construction of the road.

It has doubtless been observed, that another reason for the small cost of this road, is, that it was built with absolutely no profits to middle-men of any character.

Of course, a great many roads are built cheaply by doing very little work, and doing that in the cheapest and worst manner possible, but this road was not one of that class.

The management did not pride themselves so much on building a cheap road as on doing so much good work at such small expense; and I think that, should any one ever rebuild this road, he will not have a puzzle similar to Mr. Whittemore's.

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Vol. XXIII.—September, 1890.

THE TURN-TABLE ON THE MAIN TRACK OF THE SILVERTON RAILROAD IN COLORADO.

By C. W. GIBBS, M. Am. Soc. C. E.

WITH DISCUSSION.

The Silverton Railroad is a short line but 17.5 miles long, and has the reputation of being the steepest (5 per cent. grade), the crookedest (30 degree curves) and the best paying road in Colorado; and is owned by one man, Otto Mears. It also has a turn-table on its main track, and it is the purpose of this paper to describe it and explain why it was so placed.

This road leaves the Denver and Rio Grande at Silverton, and runs over a divide 11 113 feet above sea level, then down into the rich mining country beyond. The country is very rough and rugged, and in order to reach the town of Red Mountain it was necessary to run up on a switchback, as no room for a loop could be found. A wye was, therefore, built, and the engine could be turned while the train stood on the main track. The engine was thus placed ahead of the train, only the train is pulled out of the station rear end ahead. It runs thus till the

turn-table is reached. The train is stopped at a point marked *A*, Plate XXII; the engine uncoupled, run on to the table, is turned and pulled up to a point near *B*, where it is stopped. The train is then allowed to drop down to the turn-table and the engine backed on to it. In coming up from Albany the train is stopped on the down grade between the summit at *B* and the table; the engine is taken off, turned on the table and run up to about *A*; the train is then allowed to drop to the table as before and the engine backed up and coupled on, taking not over five minutes in going either way.

The reason of putting the table in was that there were no mines to the east of Ironton as shown on Plate XXI, but between the turn-table and the loop there were several that it was very desirable to reach, and the side hill is so steep that it is impossible to make a loop on it.

This table is the source of a great deal of comment from tourists, of whom there are many during the summer months, as it is on the line known as the "circle," so extensively advertised by the Denver and Rio Grande Railroad.

The road is used both for a freight and passenger road, and as before mentioned, is the best paying road in Colorado, two engines being kept busy hauling ore to Silverton from the Red Mountain district.

The object of writing this paper was to describe what the author thinks is quite a novelty, being the only turn-table that he has ever heard of which is used upon a switchback in this manner, and where the grades are adjusted as they are to let the train run by gravity on the table from both ways.

Plate XXI is a print from a photograph of the map filed in Washington, and is about 9 000 feet to the inch.

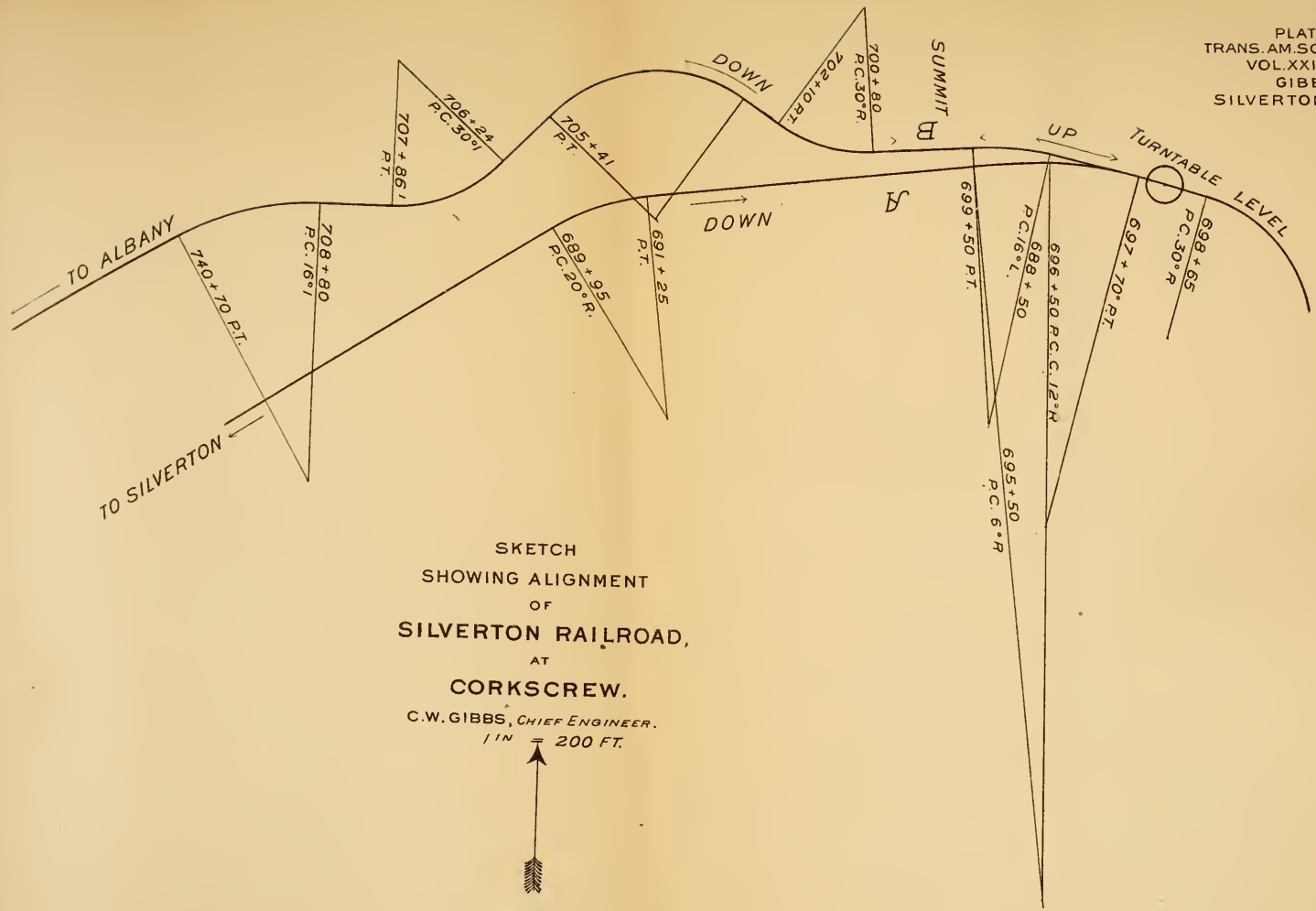
Plate XXII is an enlarged sketch of the line near the turn-table.

DISCUSSION.

J. Foster Crowell, M. Am. Soc. C. E.—It occurs to me that the use of this turn-table being simply to turn the engine during transit, while the train waits, and, moreover, as the service is a special one on a spur line, it would have been better to obtain an engine capable of running in either direction and not requiring to be turned, rather than resort to a turn-table in the main track which contains an element of danger as well as of delay to the traffic. The device, however, is an ingenious one to meet the peculiar conditions of line; and if experience with it proves satisfactory, there are other problems on a larger scale relating to change of direction in mountain location that it may help to solve.

C. W. Gibbs, M. Am. Soc. C. E.—If a special engine had been procured, as Mr. Crowell suggests, it would have been at an extra expense, owing to the limited number wanted; and even with a special design, it might have been difficult for any engine to have backed its load over so steep a grade and such sharp curves without more danger than was suggested there might be at the turn-table. The delay to traffic amounts to nothing, for there are no competing lines, nor do I expect there ever will be. The turn-table has now been in actual operation every day since June, 1889, and no accident has ever occurred.

PLATE XXI.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXIII. No 450
GIBBS ON
SILVERTON RAILROAD.



SKETCH
SHOWING ALIGNMENT
OF
SILVERTON RAILROAD,
AT
CORKSCREW.

C. W. GIBBS, *Chief Engineer.*

1 IN = 200 FT.

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Vol. XXIII.—September, 1890.

LITTORAL MOVEMENTS OF THE NEW JERSEY COAST, WITH REMARKS ON BEACH PROTECTION AND JETTY REACTION.

By LEWIS M. HAUPT, M. Am. Soc. C. E.

WITH DISCUSSION.

RESULTANT DIRECTION OF DRIFT.

In a protracted study of the changes occurring at the various inlets on our alluvial coast, with a view to determine the resultant of the forces producing them, the author came to the conclusion that, whatever the cause, there was in many cases a determinate direction to the movement of littoral drift; that it was this drift which formed the most potent element in obstructing the navigable channels of inlets, and that but little aid could be obtained from the use of parallel or convergent jetting for removing bars by the scour from concentrated ebb currents.

PREDOMINATING FORCES CAUSING MOVEMENT.

It was, moreover, believed that of the various forces, winds, waves, currents and tides which conspired to mould these bars, it would be

found that the unceasing action of the breakers upon a receding shore line, as affected by the direction of the flood tide and littoral currents, was far more effective than the "prevailing winds," and offered the only satisfactory solution of many phenomena which were in direct violation of the wind-wave theory.

As it is of great importance in maritime works, that the direction and quantity of the resultant movement of shingle should be known, and as this "flood component" theory advanced by the author has been severely criticised, it has induced him to collect such data as might be found bearing upon the subject, and to make further observations along extended portions of our Atlantic coast line, where it might reasonably be supposed the "prevailing winds" were nearly or quite constant in direction. For this purpose the coast of New Jersey from Sandy Hook to Cape May has been selected for investigation. Here, if the wind-wave theory be correct, and especially if the movement be due to the more violent gales from the north-east, as asserted, the resultant travel of the beaches should be uniformly to the southward; or, from whatever quarter the maximum wind force may proceed, the sands should doubtless move to leeward, but always in the same direction for wind from the same quarter. It would seem anomalous, therefore, to find, as a matter of fact, that from a certain point on this coast-line the inlets and beaches are traveling northward and at other points southward. But when it is noted that these directions correspond with those of the flood tide along this shore, it indicates more than an accidental coincidence.

Angular Wave Action.

Moreover, hitherto but little stress has been laid upon the angular movement of waves upon the shore, whereby a zig-zag motion of the solid particles is produced; but it has been freely asserted that, "whatever may be the direction of the waves in deep water, on approaching the shore they will break normally."

These prevalent opinions are, it is believed, the source of erroneous conclusions as to the best method of treating problems relating to harbor improvements on alluvial coasts. In the course of this paper the writer will submit some additional facts bearing upon these points, tending to show that in general the waves break at an angle and produce a littoral transportation of material in the direction of the flood

movement, and will propose a plan for creating and maintaining deep water channels over the bars obstructing harbor entrances.

Reaction *vs.* Velocity.

The writer desires also to call attention to a commonly accepted dogma concerning the influence of velocity in producing a scouring away of bars, and to emphasize the fact that this effect is not due to velocity so much as to the reaction caused by an obstruction, which develops the potential energy of a stream and renders it kinetic. Numerous instances may be cited where the bottom velocity is apparently far greater than that necessary to move the material composing the bed, yet the water flows rapidly over the bars and no motion of the silt takes place. For example, it is said that at the Narrows of the New York entrance, a mean ebb velocity of two feet per second produced a depth of 100 feet, yet instances are frequent where a like velocity under similar circumstances does not scour below 12 feet. The conclusion is irresistible that there must be some other agency at work than mere velocity. In this instance investigation has shown that it is the flood tide and not the ebb, which being compressed through the gorge, has produced the great depth mentioned, and that this depth is not that due to mean velocity but to the eddying or boring action of the millions of tons of tidal waters urged through this gorge during the flood tide. Here the bottom movements are northward for a large percentage of the time.

Normal Scour in Tidal Inlets.

In tidal waters, where there are periodic currents in both directions passing through a gorge or inlet, there will be a scour in both directions, and a deep hole or pocket will be found between the two spits which flank the inlet. The material from this pocket is carried in by the flood, and forms the inner bar, with its lateral channels and middle ground, which latter may at times project above the surface and form islands. The ebb also carries some material to the outer bar, the bulk of which is composed of littoral drift, which the ebb moulds into bars and channels.

It should be observed that in inlets dependent upon tidal movements, where the fresh water volume is inappreciable, the resulting natural

form is that just described, and the limit of depth in the channel over the outer bar seldom exceeds 12 feet below mean low water.

Erroneous Deductions.

Any plan therefore which attempts to follow out the analogy of nature, and aims at producing a contraction upon the crest of a bar, by parallel or converging jetties in pairs, without considering the inner and outer shoals which must result from such construction, is radically defective. This method only aggravates the evil by creating two gorges, or a long channel, with greater frictional resistance to the tidal prism, the only volume available for scour, and requires heavy annual expenses for dredging and maintenance. It must be remembered that this criticism applies to those cases where the scour is produced solely by tides unaided by any considerable land drainage, as at Nantucket, Manasquan, Galveston, Aransas and similar ports.

Sandy Hook.

But to return to the consideration of the special locality chosen for examination and study. The largest and deepest entrance on the Jersey coast is that at New York Bay, where the normal depth is 23 feet M. L. W., and the width of the gorge between Sandy Hook and Coney Island is about 5 miles. The peculiarities of this bar with the abnormal slough, 52 feet deep, lying along its crest and directly across the path of the currents, and a method of utilizing the powerful natural resultant there present, have been described already in a paper, by the writer, read at the Buffalo meeting of the American Association for the Advancement of Science (August 20th, 1886). The main point to which it is desired to direct attention in this connection is that, notwithstanding the great volume and velocity of the currents across the head of the spit, Sandy Hook has continued to advance northwardly until it reaches out across the bay a distance of about 5 miles. The Shrewsbury and Navesink Rivers, which until recent years found an outlet across the neck of this spit of sand into the Atlantic, have been dammed up by the northward movement of the drift and diverted into the bay. During heavy storms it occasionally cuts through the beach, but only for a time. As to the rate of growth of the Hook, there is much diversity of opinion. Professor Alexander Dallas Bache said, in his report of the United States Coast Survey for 1856: "Within a century it has increased a mile and a quarter;" while the Board of Engineers, in Reporting on Gedney's Chan-

nel, say, concerning the Hook: "In the last one hundred and forty-five years its length has varied only about 2 700 feet, sometimes increasing and sometimes decreasing."*

With reference to the origin of its material there is now no question. Special surveys were made by Professor Henry Mitchell to determine this mooted question, and he reported, "That it turns out that this growth of the Hook is not an accidental phenomena, but goes on regularly and according to determinable laws. The amount of increase depends upon variable causes; but the general fact is that it increases year by year, and the cause of this is a remarkable northwardly current * * * along both shores of the Hook. * * * For more than seven hours out of the twelve there is a northwardly current through False Hook channel. This northward current runs inside for eleven hours out of twelve. It is the conflict of these two northwardly currents, outside and inside, and the deposit of materials which they carry to the point of the Hook, which causes its growth."—U. S. C. S. Report, 1856.

Many interesting phenomena may be developed from the comparative study of the numerous reports on this typical sand spit, but the subject is too comprehensive for further elaboration herein.†

In passing, and before leaving this site, it will be *à propos* to note that Norton's Point, at the western end of Coney Island, has been extended westerly by the similar action of natural forces.

Proceeding southward from Sandy Hook, it will be observed that the many bays which at one time indented the coast, are virtually closed by the beaches which have grown across their outlets, and are now transformed into lakes as at Elberon, Deal, Asbury Park, Ocean Grove, Ocean Park, Lake Como, Spring Lake and Sea Girt.

The prominent exceptions in this 25 mile stretch of coast are Shark River and Manasquan, and these deserve a brief examination to illustrate the cyclic changes which are so characteristic of similar localities.

Shark River Inlet, 20 Miles South of Sandy Hook.

This river separates Ocean Beach from Key East. The inlet is very fickle as to position, and at low tide the water-way is barely 100 feet

* Ex. Doc. 78, 48th Cong., 2d Session, H. of Rep., p. 12.

† For further information reference is made to the Reports of the United States Coast Survey and those of the Chief Engineers, as well as to the Proceedings of the American Philosophical Society for March, 1889, containing a paper by the writer, entitled: "Discussion on the Dynamic Action of the Ocean in Building Bars."

wide. Immediately inside the beach it opens out into a long narrow lake, forming a large tidal reservoir, extending inland several miles, with depths varying from 1 to 3 feet.

"The bar on the outside is dry at low water, and is slowly working to the northward in a manner characteristic of similar ones along the coast." * * * "The average rise and fall of the tide at the inlet is 1.4 feet." The rise and fall of the tide in the ocean at this point is about 4 feet. At the railroad bridge, about 500 feet inside, it is 0.7 feet. The velocity of the current in the inlet is 5 miles per hour. The conclusion seems to be warranted that the flood current is considerably stronger than the ebb.*

As there is no commerce it was not deemed advisable to attempt any improvement of this inlet.

From the relative tidal fluctuations, as quoted, it will be observed that the contraction of the entrance by the extensive spit of sand very materially interferes with the filling of the inner basin by the tides, while the short duration of the flood as compared with that of the ebb causes a rapid scour inward of the beach sand, which is immediately dropped on the extensive middle ground flat, just inside the inlet.

On September 5th, 1889, the writer made a sketch, in plan, of this inlet, to illustrate the extent and direction of its movement. The year before it was reported to have been closed, and the residents of Ocean Beach cut a channel across the southern end of the spit, 700 feet south of the present channel. This opening was maintained by the currents, but has progressed northward to the position shown in Fig. 1, Plate XXIV.

One of the highest tides and most severe northers occurred a few days after this sketch was made, viz., September 12th to 15th, succeeded by other storms from the same quarter, furnishing an unusual opportunity to verify the truth of the prevalent wind-wave theory, when the travel of the beaches should have been southward or to leeward. The writer therefore visited this site again soon after, and found the channel had moved fully 4 rods north of its previous position on September 5th, which was a result just the reverse of what should be expected, if the wind-waves are the forces mainly relied upon to produce these effects.

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* Extracts from Report of Assistant A. Doerflinger. Report Chief of Engineers, 1881, vol. 1, page 731.

Manasquan River Inlet.

"This opening is situated in Monmouth County and is about 25 miles south from Sandy Hook. The drainage basin embraces about 80 square miles. With a view to its improvement a survey was made in September, 1878. It showed a depth on the bar at low water varying from about 1 foot to 18 inches. The tidal range is about 2.4 feet, and the inlet was slowly working to the northward by wearing away the main beach on the north side of the lowest bend, the south beach following."*

"Subsequent to the heavy storm of October 25th, 1878, the inlet broke out nearly in the position of the proposed jetties. This action is entirely characteristic of the habit of the inlet. Its normal and effective position is the present one, viz., on the prolongation of the inner neck which does not vary. At such times the depth on the bar is from 3 to 4 feet or more. The inlet does not, however, remain in this position, but gradually forms the *S* shown on the chart from the northward movement of the sands along the front beach."

"The main stem of the *S* lengthens by degrees and the inlet advances northwardly, eating its way through the beach, but always maintaining its connection with the inner neck and constantly becoming more obstructed until its discharge nearly ceases. On one such journey northward the inlet reached a point near the present life-saving station, and in consequence, the river was filled with nearly stagnant water, giving rise to disease and causing great mortality among the fish. The inhabitants of the neighborhood combined and dug the inlet through at the proper place with the result of immediate relief. Such extreme cases are rare and the inlet after traveling a few hundred yards, generally under the influence of strong westerly winds, breaks through without artificial aid. The intractibility of the inlet has always been the most serious obstacle to the use and improvement of the river." * * * "The following project for the improvement of the Manasquan is respectfully submitted:"†

"To construct the piers of the form and dimensions, and in the positions shown upon the chart (see Fig. 2, Plate XXIV), for the purpose of controlling the inlet and concentrating the scouring action of the ebb."

"Assuming that the mean rise and fall in the lower basin will be 1½ feet, it is estimated that the discharge will be sufficient to maintain a low water depth of 10 or 12 feet in the middle of the gorge, and 6 or 7 feet on the bar, which will form exterior to the outer wings.

"This bar will be exposed to the action of the littoral currents due to the northwesterly direction of the tidal wave and the effects of winds, and will probably rise to a height above that given. * * *

"The axis of the jetty is in near conformity to the general direction of the outflow, and its position is such as to admit of the inner wings making equal angles with it while reaching points on either side where the ends will not be endangered. The outer wings also make equal angles with the axis, and as nearly as possible, without being too much inclined to the shore, face the tidal wave."‡

The total estimated cost of this improvement is $52 120. This project was referred to the Board of Engineers, who made both a majority

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* Reports Chief of Engineers.

† Vide Report of Colonel William Ludlow to Chief of Engineers, 1879, page 463.

‡ *Ibid.*

and minority report in April, 1880. In the former it is said, *inter alia:* The flood-tide impinges upon the shore from a direction south of east, and the channel wanders in this direction, growing constantly more crooked until it becomes partly, and in some cases entirely, choked. These are familiar features common to sandy beaches, showing a tendency of the inlets to move in the direction of the predominant drift. "In such cases the Board sees no permanent remedy except to transfer outwards and fix the mouth of the inlet by artificial works, terminating at depths where the resultant forces of the waves and currents would prevent the formation of a hurtful bar."

The Board recommends that two dikes should be built out to sea, with a narrow opening between them, and that the north one should be the first built. In the minority report Colonel Macomb states that "the lengthening of the contracted channel will decrease its capacity to admit the tide, and therefore diminish the inner tidal prism, upon which alone the channel must depend."

In September of the same year the data and reports having been referred to Colonel N. Michler, he reported, indorsing the views of the Board, and adds: "The exterior lines of piers beyond the beach will be of an experimental nature; the effects of storms upon them will prove their stability, and the movable drifting sands will in one way or another decide the problem as to the formation of a new bar across the artificial entrance to the inlet. Owing to the absence and untried character of any similar work along the Atlantic coast, * * * a safe estimate cannot be made as to the probable final cost."

While these extracts show the uncertainty attending the methods and anticipated results they leave no question as to the existence of strong northwardly tidal currents, which are recognized as the principal agencies in shifting and closing this inlet, yet the method proposed for "opening a direct channel-way across the beach, and protecting the same from the flow of the tidal currents between the river and the ocean," appears to be designed to utilize the ebb scour rather than to arrest the exterior sand movement and admit the flood tide freely.

Under the approved plans a contract was entered into July 1st, 1881, for the north jetty first, and the construction of the south jetty was left to the discretion of the officer in charge, then Major G. L. Gillespie. The line of this work is shown in Figure 3, Plate XXIV. Its position is such as to catch the flood currents charged with littoral drift and deflect them

into the inner basin, where, in consequence of the increased section and reduced velocity, new shoals immediately formed, while the reaction along the dike itself produced so much scour as to make protecting spurs necessary. These inner shoals operated to deflect the ebb currents over a different path from that by which they had entered, so that they failed to remove these deposits, and the basin rapidly deteriorated. During this part of the construction the depth on the bar was 4 feet, but the work was said to be of so experimental a nature as not to warrant any prediction of final results. "To extend the north jetty 255 feet seaward to its full projected extent and to build 905 feet of jetty on the south side will require, it is estimated, the sum of $40 000.* The length of the north jetty, at the close of the fiscal year 1882, was 1 515 feet. On September 26th, 1882, a contract was made with S. A. Kelly for a portion of the south jetty, and operations were commenced in November. It is located 200 feet from the north jetty, and begins at a point 100 feet inside of the end of the north jetty, extending inland 475 feet, and diverging so as to concentrate the river currents and to "prevent the cutting of the beach behind the jetty. The inshore part is still too short, and should be extended 300 feet inshore towards the meadow to remove all apprehension of the currents making a breach through the South Beach."†

The condition in which the work was left is shown in the plan for 1887 (Fig. 3), at which time the river had cut a channel behind the south jetty, leaving it entirely detached from its supporting bank, while the former bed between the jetties was being rapidly filled by sand carried in by the flood tides. In August, 1889, when the writer visited this locality to observe effects, the entire space between the jetties was found to be filled with sand, as shown in Fig. 4, Plate XXIV. The river had abandoned its former bed, and had been deflected to the southward several hundred feet, leaving the jetties, to a great extent, buried under the drift. The outer end of the south jetty was being destroyed by the cross-currents, and the inlet was almost closed. The general conditions are more clearly illustrated in the panoramic photograph taken in May of this year (Plate XXIII).

The history of this case is given in some detail, since it is one of the most instructive examples of the injurious effects of twin jetties to be

* Report Chief of Engineers, 1882, page 702.

† Report Chief of Engineers, 1883, page 584.

found unrecorded, and because it is probable that in view of an item in the present river and harbor bill the work done here may soon be removed, and the experience be lost to the profession.

The item reads: "Improving Squan River, New Jersey, by the removal of obstructions placed by the Government at the mouth of said river, if, in the discretion of the Secretary of War, the same should be done, $2 000."

The amount expended upon these jetties was $39 000. The north jetty was built first, and catching the flood tide, charged with sand, deflected it, precipitating its load in the middle ground, seen in Fig. 2, which the ebb was unable to remove. Thus these works were found to be unable to concentrate the ebb currents which were relied upon for scour, and it is also manifest that they formed a pocket for the littoral drift carried in by the flood.

In many respects these results may be taken as a type of those which will ultimately follow the completion of the jetties at Galveston on the present plan of jetties divergent to the flood movement, and for the completion of which $6 200 000 is requested. In the above plans, as elsewhere, the object of the designers has been to concentrate the ebb scour, while, in the writer's opinion, a much better result would have been obtained had the works been designed to oppose the sand movements and defend the channel from the encroachments of the flood and littoral currents, while freely admitting the tide.

As these outer bars are generally built by drift and wave action, if the agencies which maintain them can be diverted or partially neutralized and the ebb assisted in its escape, there should result a better crossing on the bar. The general principles which have been formulated to accomplish this result are so fully stated in a paper on "The Physical Phenomena of Harbor Entrances,"* that they need not be repeated here, but may be summarized as follows: 1. Keep out the littoral drift; 2. Let in the flood tide; 3. Conserve the ebb energy over a limited sector of the bar, and 4. Maintain a continuous reaction upon the ebb currents.

It will be seen that in many cases, particularly where there is a resultant direction to the movement of the drift, all of these conditions can be fulfilled better by a single jetty or breakwater of proper form and position than by two, and at less cost. Thus it will appear that this

* See Proceedings of American Philosophical Society, 1888.

plan is designed to apply the "ounce of prevention," rather than the "pound of cure."

But, to continue our investigations of the coast line, let us proceed southwardly from Manasquan.

The Node.

Hitherto it will be seen that the littoral drift is northward, but at the next inlet, and, so far as authentic records show, at all the subsequent breaks through the sandy cordon defending the Jersey coast, even to Delaware Bay, the prevailing movements are southward. This opposition of movement has been supposed by some authorities to be due to the shelter or lee afforded from the north and northeast storms by Long Island. But this explanation is unsatisfactory, since the energy of the storms is quite as manifest at Sandy Hook as at Cape May. Neither is there any change of direction in the coast line, producing a salient which would divide the tidal movements as at the outlying primary nodes at Hatteras and Nantucket at the extremities of this middle bay. That there is a strong littoral current, however, is indicated by the absence of any breaks through the low sandy peninsula known as Island Beach, which extends from Bay Head to Barnegat Inlet, about 23 miles; and that the drift is southward is clearly shown by the movement of this inlet, to the south about one-half a mile, between 1840 and 1874, or in thirty-four years. (See Plate XXV.)

The approximate location of this secondary node, as distinguished from the primary ones above mentioned, may possibly be indicated by the greater slope of the foreshore between Lavallette City and Point Pleasant, where the ten-fathom curve approaches to within 1¼ statute miles of the beach instead of remaining from 4 to 5 miles off-shore, as elsewhere along the coast.

From Long Branch to Barnegat Inlet the trend of the coast is south 14 degrees west in almost a straight line, being slightly concave. At this latter inlet it deflects 25 degrees further west, to south 39 degrees west, making an abrupt change of direction, and thus furnishing an opportunity for the waters of Barnegat Bay to effect an escape by the break in the continuity of the littoral currents caused by the angle.

The movements at this inlet are so clearly shown by the comparative sketch herewith as to require no amplification. (Plate XXV.)

From Barnegat we traverse Long Beach for about 23 miles to New Inlet opening into Great Bay and Little Egg Harbor.

This is so named to distinguish it from "Old Inlet," which existed prior to 1846 and which is shown on the charts as being navigable for vessels drawing less than 5 feet of water. This inlet, which was one mile wide, has been entirely closed by the drifting sand, and Long Beach is now united to what was Tucker's Island, but is now Tucker's Beach, while the channel across the bar is about 5¼ miles southward of its old position. Here is found the characteristic flood channel lying directly under the lea of the Northern Spit and the long ebb channel stretching off to the south where the bar building forces are weakest. (Plates XXVI and XXVII.)

Similar features are observable at Absecon Inlet, where protection works have been necessary to protect the Light House on the Southern Spit from the encroachments of the sea, and also at Great Egg Harbor, Corson's, Townsend's, Hereford's, Turtle Gut and Cold Spring Inlets to Cape May, there are to be found to a greater or lesser degree the same phenomena. This southward travel of drift is arrested at the mouth of the bay and forms the extensive shoals obstructing the northern flank. Much of this material is carried inside by the flood and deposited on the banks. The large bar known as Eph's Shoal, which is over a mile in length and more than half a mile off shore, has traveled westward a distance equal to its length within forty years.

These numerous inlets remain in a state of nature with vacillating, shallow channels over the bars. No attempt has been made to improve them, although, at Absecon, surveys were made and a plan designed for a harbor of refuge, but as it was estimated to cost $3 000 000, it was not thought, in view of the great cost and the uncertainty as to results, that the expenditure would be justified.

The proposition to construct a ship canal from Philadelphia to the Atlantic also lent importance to the question of creating a harbor at this point, but as the time consumed *via* canal would be about equal to that *via* river and ocean, it does not enter as an important factor. If, however, the channel can be fixed and the sand movements arrested by a single breakwater located on the northern flank of the ebb *thalweg* and open at the shore end to let in the flood tide freely, the cost may be reduced to one-half or less and the depth be greatly improved.

Practical Applications.

These phenomena are not peculiar to the New Jersey coast. The southern shore of Long Island furnishes another instance of a secondary nodal point from which the littoral drift moves east and west. The writer has not been able to collect sufficient data to locate this point even approximately, but it will be found to lie somewhere to the east of Petunk, Long. 72° 30′. Again, on the sea front of Cape Cod peninsula a point of division of the tides will be found south of Truro, whence the drift proceeds in opposite directions. These movements do not appear, therefore, to be controlled by prevailing winds, but by tides, and they form most important factors in the design of any plans for improving harbor entrances.

That they are not mere speculations of the writer will be seen by a few extracts from the interesting discussions of the Institution of Civil Engineers upon a paper entitled "The East Coast" (of England), by J. B. Redman, in January, 1864.* Sir John Rennie said:

"Without knowing and carefully studying the operation of these natural phenomena, it would be in vain for the engineer to attempt to design any work which might be intrusted to his charge for the improvement of rivers or harbors along such a line of coast, for most probably the works would prove to be abortive and the money expended upon them would be in a great measure thrown away."

He then instances the case of "the celebrated Smeaton, who essayed to improve the harbor of Rye by making a new mouth to the westward of the old one, which was effected in the first instance at a great expense, and a few years subsequently was filled up by the beach traveling from the westward." * * * "Another important example might be seen at Harwich, where a fine natural harbor had been materially injured by the construction of a pier on the wrong side of the entrance."

Captain Burstal of the Royal Navy, said that "from his observations both on the east coast and on the south coast, the beach did not travel with the prevailing wind but with the prevailing sea."

Mr. J. Scott Russell, V. P., remarked that "the direction of the prevalent sea was much more important in the formation of beaches than the direction of local and prevalent winds."

The Astronomer Royal said: "That in a bay, whatever was the tendency of the prevailing wind, or prevaling surf, the drift of the pebbles was always inward from the horns of the bay to the center of the bay."

Mr. J. S. Valentine stated that "he was told, as he had heard it stated in this discussion, that the travel of the shingle was in the direction of the prevailing winds. Upon investigation he found that conclusion was erroneous. He believed the travel of the shingle along that

*Vol. XXIII Proceedings.

coast was not in the direction of the prevailing winds, but was entirely independent of it, and that it was due to the great tidal wave."

Sir John Coode (now President), in explanation said "there seemed to be an impression that he had stated that the travel of the shingle was in the direction of the prevailing winds. * * * But, to show what his views were on that point, he would quote a short passage from his paper on the Chesil Bank, read in 1853, viz., the fact of the largest shingle being found, at what has been called 'to leeward,' is well known in the neighborhood; this term 'to leeward,' must, however, if applied to shingle, be clearly understood as relating to the quarter whence the heaviest seas proceed and not to the winds."

The above quotations are adduced as additional testimony from some of the most experienced foreign maritime engineers in support of what has been called the "theory" advanced by the writer, as to the predominating influence of the wave movements as affected by the angle of approach and by the flood tides, rather than by prevailing winds, and which has led, it is believed, in this country as elsewhere, to the location of jetties on the wrong sides of inlets and to the use of two where one would have been more effective.*

Shore Protection.

Such are a few of the practical conclusions deduced from this study of the littoral movements of the beaches along the New Jersey coast, but there are also other effects produced which have not yet been mentioned —such as the degradation of the alluvial bluffs that characterize the coast in the vicinity of Long Branch.

Of late years the erosion here has been the more noticeable since it has frequently made serious breaches in the boulevard surmounting the crest. It is not long since there were cultivated fields between this drive and brow of the bluff. While no comparative charts are available as a basis to determine the rate of advance, it is believed from the best information obtainable to have averaged not less than 15 feet per year, or 12 cubic yards per lineal foot per annum. To prevent the destruction of valuable property, the private owners have recently revetted the face of the bluff with sheet piling or dykes, or in some cases spur jetties to catch the drift and resist the waves. One of these short spurs, showing the embayed sand drift on its southern flank, is shown in Plate XXVIII, furnishing another proof of the resultant direction of the movement.

* For additional testimony on this point reference is made to the Discussion on the Dynamic Action of the Ocean in Building Bars, published by the American Phil. Soc., March, 1889. Phila.

It also shows the angularly approaching breaker moving northwardly while the wind at the time was fresh from the northwest. The photograph was taken just after the severe norther of November, 1889. The jetty is normal to the beach.

Another view (Plate XXIX), taken earlier at Atlantic City, reveals more clearly the southward roll of the breakers at that locality. These are instructive as showing that the waves do break at an angle upon the shores and may therefore readily transport materials in the direction of their motion over a zig-zag path, as has been occasionally affirmed, although not generally accepted.

Recently mass meetings of citizens of Long Branch have been held to devise plans for the protection of the site, and various tenders have been submitted. In addition to those already referred to was one to construct a low water wave breaker, parallel to the shore, to resist the sea, but thus far nothing more has been done than the independent and local applications made by each owner for himself.

A systematic plan of deflecting jetties would, it is believed, be more effective and much cheaper. But it is not the purpose of the writer to encumber this paper with further details as to shore protection. A few instantaneous views (Plates XXX and XXXI), taken after the storms of last fall, will sufficiently illustrate the methods pursued, showing great variations from the vertical to the inclined revetment, but all supported on the face of the bluff, here about 20 feet high. Some of these the waves easily mount, falling over on the drive to its injury. The general appearance at that time resembled a series of "back-yards" strewn with rubbish.

Detached or Insular Wave-breakers.

There is one phenomenon, however, frequently observed in natural formations, which has not been utilized as a means of protection, which will be interesting to the Society—and that is the tendency which exists for the formation of shoals in the lee of any outlying obstruction, where waves or currents passing around the ends of an island meet in its rear and build up bars.

Examples of this form are to be found in the numerous Presque Isle formations along the coast and in the Great Lakes. The low sand spit joining a rocky headland, so familiar to all maritime engineers, is doubtless the result of current and wave deposit along the neutral axis connecting

the original island with the strand. For instances, see Richmond's Island Harbor and vicinity, Maine (Plate XXXII); Boston Bay (Plate XXXIII), contains several well defined examples, as at Marblehead Neck, Nahant and Little Nahant, Winthrop Head and Nantasket Beach, with its chain of former islands; or a still more remarkable case of a double beach formation may be seen in the Peninsula of Giens, off the southeast coast of France near Toulon, where a lake (Manarre) is inclosed by the nearly parallel beaches connecting the headland with the main coast. (See sketch, Fig. 5, Plate XXIV.)

In certain localities this tendency to deposit may be utilized to advantage for shore protection—but it may also become an evil where deep water is desired, as for harbors of refuge.

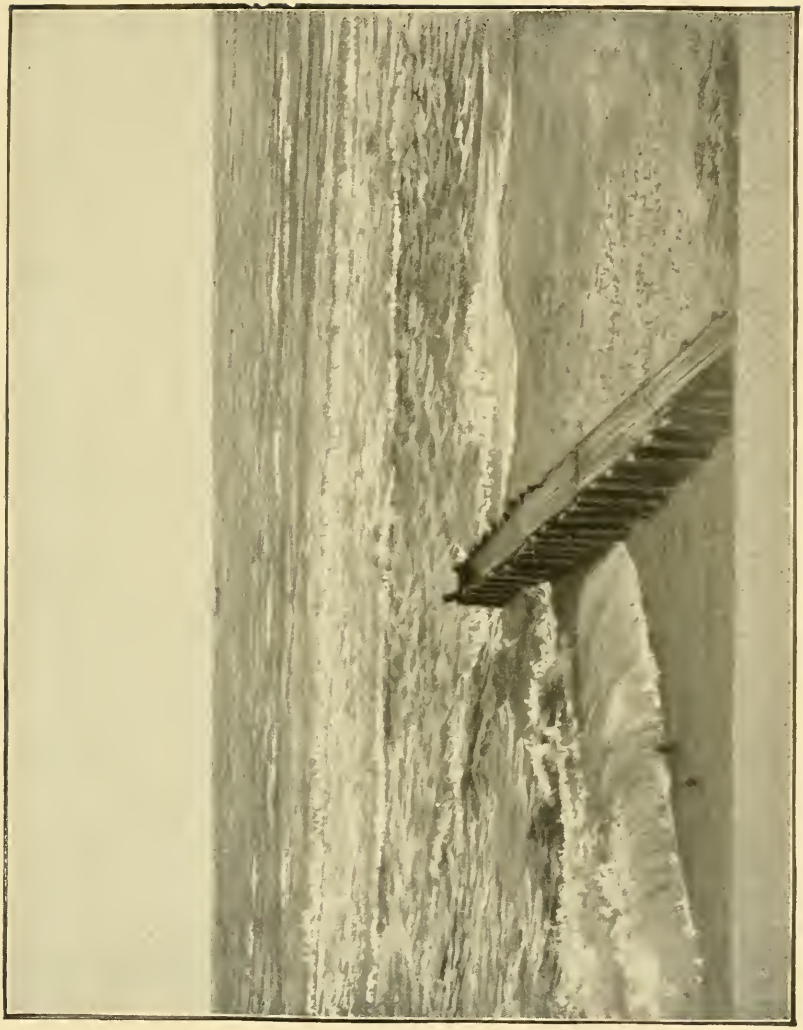
The most notorious instance of deposits resulting from the creation of an artificial island is to be found at the Delaware Breakwater, where, since its commencement, in 1828, the 18 and 24 foot curves have advanced from the shore to the breakwater more than 1 mile, while in the center of the harbor the low water depth has shoaled to 14 feet, with a mound having only 10 feet near the west end of the breakwater.* The harbor is useless for deep draught vessels, due to the checking of the currents by these structures, and a new one is now contemplated on "the Shears."

Reaction.

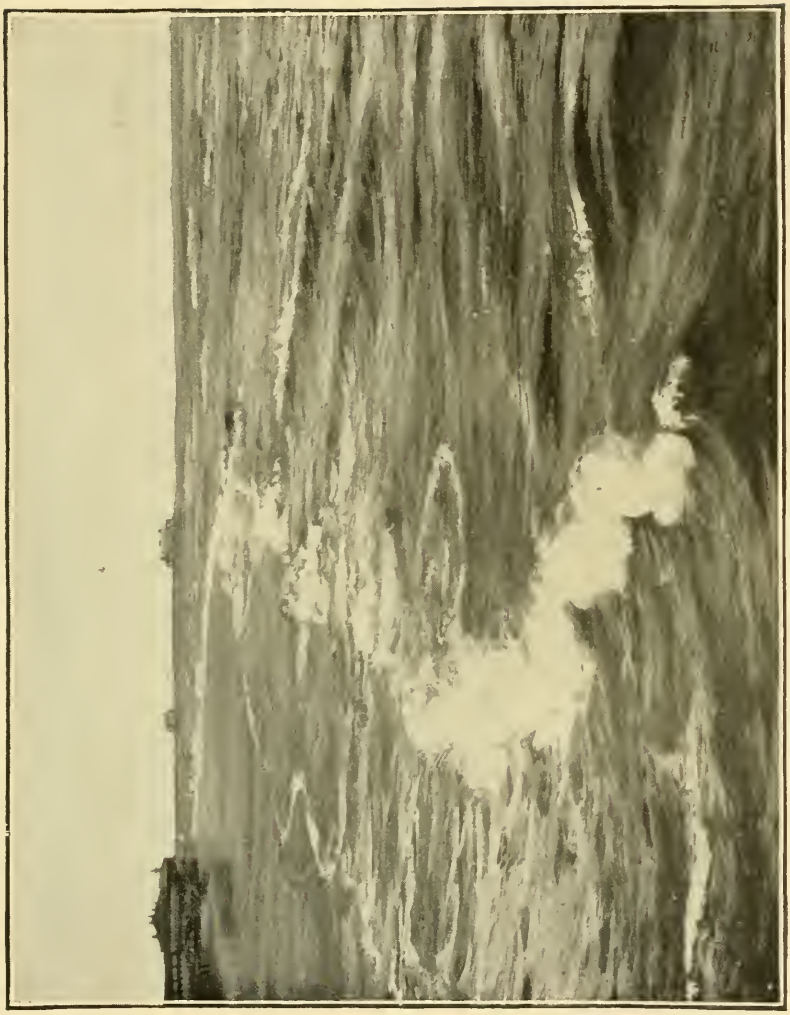
This paper would be incomplete were no allusion made to the reasons why some jetty works are more successful than others and require less dredging for maintenance.

Aside from the general physical conditions which determine the practicability and probable utility of jetty work, it is of the utmost importance to determine at the outset the proper width to be given to the entrance. This question has already provoked an interesting discussion before this Society, but evidently without reaching any very definite conclusions, since there is a project now under contract, in tidal waters, where it is expected to obtain a 30 foot channel, 5 miles long, with jetties 7 000 feet apart. The works at the mouth of the Maas were found to be too far apart, as first built, and have been contracted; those at the South pass, originally 1 000 feet apart, have been reduced to 600 by

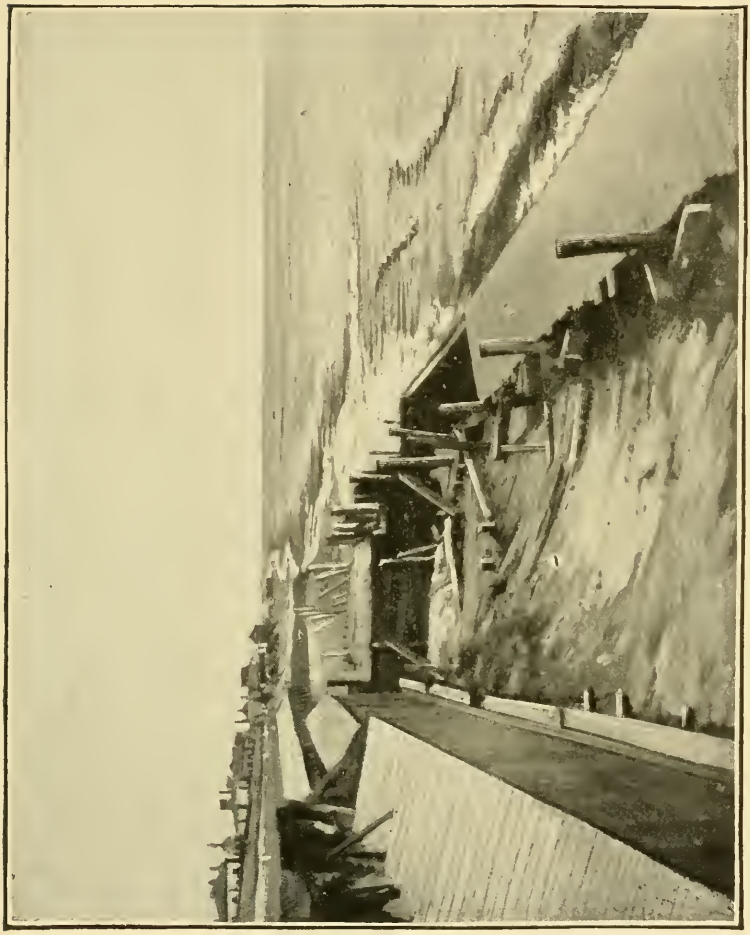
* See "Harbor Studies," by the Author, *Proceedings* of the Engineer's Club of Philadelphia. Vol. V, No. 4, page 285. April, 1886.



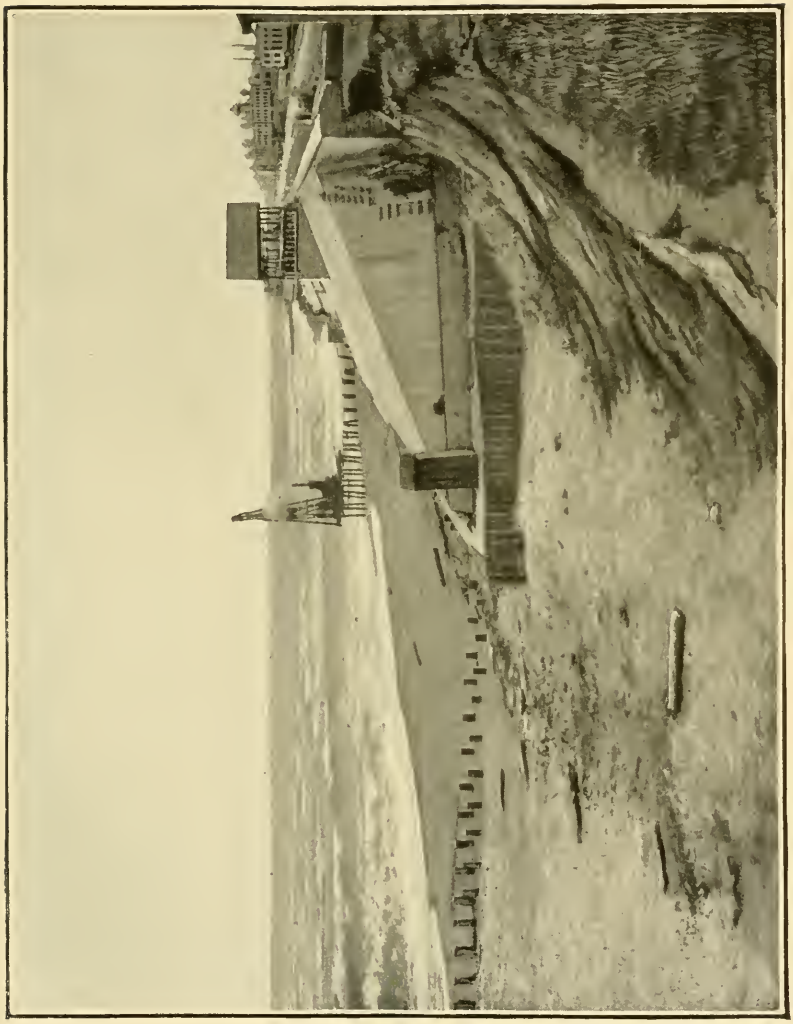
SPUR DYKE AT LONG BRANCH, N. J.



ANGULAR WAVE MOVEMENTS AT ATLANTIC CITY, N. J.



SHORE PROTECTION, LONG BRANCH, N. J.



SHORE PROTECTION, LONG BRANCH, N. J.

spurs and lateral contraction. The jetties at Sabine Pass are conceded to be much too far apart to produce the desired result, and so, as in many other cases, it becomes fundamentally important to determine just how far apart the jetties may be placed that they may co-act and produce the maximum effect with the available prism of discharge. The answer to this question must be found in the amount of reaction which the jetties can be made to develop—depending upon the resultant volume and velocity of the fluid prism. It is well known that a trough is scoured out by currents flowing parallel to obstructions, and that deep holes or pools are produced by concave bends, but it does not appear to be recognized that this is the cause of, as well as the limit to, the beneficial action to be derived from jetties. For example, let it be assumed that two jetties are placed a mile apart. The fluid prisms adjacent to the jetties will be materially affected by the frictional resistance of the walls so as to eddy and bore upon the bottom, thus cutting two channels, one on either flank, while the central prism will not be materially affected, and there will be left a middle-ground, ridge or shoal. As the space between the jetties is contracted, although the cross-section diminishes in the same ratio, the amount of the reaction of each jetty remains practically constant and the middle ground is cut down, until, when the jetties are brought so close as to co-act, it disappears altogether and there will result a normal and self-maintaining channel.

If a jetty or obstruction be placed obliquely to a current the energy of the latter is suddenly developed by the resistance and change of direction and a deep hole is bored in the bottom.

The enfeebled current then creeps along the side of a straight jetty, dropping or refusing to pick up its load, which remains heaped up on the bar or is rolled further seaward.

Many instances of these results might be cited in practice, whereas if the jetty had been so designed, in plan, as to start from its abutment in a direction tangent to the flow, and had been made to curve gradually over the obstructing bar so as to maintain a continuous reaction along its concave face to deep water, a channel sufficiently capacious for navigation could be secured and maintained. For ordinary velocities of from 3 to 5 feet per second, the width of the channel at the 25-foot contour will be found to vary from 300 to 400 feet. Hence where two jetties may be used without seriously diminishing the volume of flow, they may be placed from 600 to 800 feet apart with good results. Where the

mouth of a stream is subject to periodic flushing from freshets and there is but little silt, which is deposited slowly, they may be even wider; but where the currents are those produced by tidal water only, then jetties in pairs will prove injurious, as ordinarily constructed, since they prevent the free ingress of the only water available for scour. In such cases a single reaction jetty, which is detached from the shore to let in the flood tide, but which maintains a continuous reaction across the bar, will be found most efficient as well as much less expensive.

The few successful instances of jetty channels to be found in the history of maritime works can readily be shown to be due to the application of the principles heretofore stated of defending the channel from the beach sand, letting in the tide, where there is one, and maintaining a reaction over the bar.

The single concave jetty at Bilbao, Spain, is probably the most successful instance of work of this description to be found on record, and its influence on the commerce of the port has been very pronounced.

The acknowledgments of the author are due to Professor T. C. Mendenhall, Superintendent of the United States Coast and Geodetic Survey, for the use of the comparative maps and charts of the New Jersey coast, and for such other data as were available, relating to this investigation, although it was admitted to be meager. To extend it one of the ablest physical hydrographers of the service, it is believed, has been assigned to the duty of making further observations in this direction.

APPENDIX.

Having learned that private parties were driving piles in Shark River, with a view to reclaim land, the writer made a flying trip to that point on June 12th, 1890, to observe the effects. The accompanying sketch, Plate XXXIV, will illustrate the existing condition of the bars and channels better than a mere verbal description. The row of piling, so far as completed, has had the effect of increasing the deposit carried in on the flood tide, and extending the inner middle ground so that it now forms an inner spit, overlapping the outer and forming between them an S, or reverse curve, with rapid cutting of the concave banks. As the jetty is extended, if slowly, so as to admit a sufficient tidal volume, it will ultimately cut a new channel across the beach to the south of the piling. The present entrance is between 300 to 400 feet north of its position last September.

Long Branch.

A visit was also paid to the beach at Long Branch and West End, where it was found that portions of the drive were poled off, in consequence of the road-bed having been washed out. The great variety of revetments existing in front of private properties, form no system and no continuous protection. In many places the junction of these different works form injurious re-entrants which throw the waves over on the drive. Some of those, put in last fall, have bulged badly, and iron rods 1 inch in diameter have been twisted like wires.

A careful measurement of the bluff gives its height as 22 feet above low water. All the spur jetties show the sand banked high on the south face, but are denuded either partially or wholly on the opposite side.

The writer was informed that an effort to assess the county for a system of beach protection having failed, it is now proposed to abandon the ocean drive and to move it to the first avenue in the rear, which is under contract for improvement.

Manasquan.

No material changes were observed at 'Squan Inlet, save a scour on the ocean side of the southern spit and a deposit on the inside, indicating a westward movement. The north jetty has prevented any material extension in that direction, but has not deepened the channel over the bar, which remains at about 1½ feet (mean low water). The south jetty has been cut away at the outer end by the rapid currents crossing its axis, and the riprap filling now obstructs the channel and makes it dangerous at low water when there is any "sea."

June 14th, 1890.

DISCUSSION.

F. Collingwood, M. Am. Soc. C. E.—The experience of all observers is that careful observation of any given locality on the sea coast will disclose a predominating movement or drift of the material composing the beach in one direction. Thus, to consider the case of the Shark River inlet (which I have observed quite closely), it is undoubtedly true that the predominant effect of all the agencies at work is to cause the inlet at this point to advance northward.

The waves, during the summer, almost uniformly approach the coast at an angle opening northward, so that both in running up the slope of the shore and in retreating, the trend of the wash is northerly, and leaves a checkerboard appearance on the surface of the sand in its northward movement. On asking one of the surfmen of the Signal Service when the inlet closed, he said usually in the fall or spring, but sometimes (as this year) in the summer. As for the cause, he said, the winds from spring to fall are more from the southeast and east, but in the winter the northeast winds hold the inlet where it is. That the winds increase or diminish the tides enormously, is a matter of common observation, and the currents due to the tides must be proportionally increased. The growth of the bar and trend of the river is, of course, the resultant of all the forces.

If the shore of the ocean were straight, and there were nothing to deflect the tidal wave from a normal approach, then there would be no tidal currents. But there are banks several miles out, and indentations, bays and inlets to be filled and emptied, so that the wave becomes one of translation, and the flood-wave will generally on this part of the coast have a northerly movement.

As to the winds, those in the southeast quadrant have a free sweep over the broad ocean, while those from the northeast cross the land (Long Island, etc.), and have a limited space on which to act. The tendency also of the conformation of the coast from New England down would be to cause the greater waves from the ocean to change the direction of their front, so that in any case the northeast winds would be less effective than the southeast in modifying the shore, and the predominant travel of the waves will be, as stated, to the northward.

The description of the river quoted from the report of the Chief of Engineers is not quite correct. The depth of the lake or bay to the west of the railroad bridge is, over considerable portions, many times "3 feet."

The railroad bridge (really it is a pile trestle) crosses at about 3 000 feet from the ocean, and it is this length which may be called the river. The bridge greatly interferes with the free flow of the tides to the

interior basin, and there are indications that the lower section of the stream is diminishing in consequence.

This year the spit shown in the sketch by the author of the paper extended afterward at least 400 feet further to the north. This increased length of the channel caused the ebb current to be too feeble to remove the sand washed over the spit at every high tide, and finally about the first of July the inlet closed.

By July 21st the water surface in the river was about 2 feet above mean tide, and a cut about 4 feet wide was made at about the point on the plan marked "Cut of '88." By night this had increased to 8 feet wide and 2 feet deep, and by the next morning to 50 feet wide and 6 feet deep. It finally enlarged to about 100 x 10 feet, with a maximum velocity of flow of at least 6 miles an hour.

Immediately the northward movement began by cutting at the north and filling at the south of the entrance, at the rate, for more than a week, of about 20 feet per day.

The stream formed a large delta at its mouth, and the greatest discharge would be first to the northeast, then southeast, then east, following always that slope which at the time was steepest.

The delta finally reached about 200 to 300 feet out, and then began to take the form of a bar, by the waves checking its further advance, and the tidal currents excavating and deepening the space inside. At the present writing this bar flares outward somewhat to the north, and the deepest water is in that direction, as is invariably the case. The travel northward of the inlet continues, but at a much slower rate.

The spit on the south now curves inward, the curvature being caused by a northward deflection of the ebb tide by the south bank, and a southerly deflection of the flood by a newly driven line of piles which extend out nearly to high water. In the September storm of last year the waves would break on the shore of the spit, and carry great quantities of sand northwesterly into the river. On the contrary, the northeast gale prevailing, blew large amounts of sand southwesterly, so as to cover fences in the vicinity with drifts.

The shore on the ocean front at Key East (now Avon) is a bluff 10 feet high, and the waves struck it with force sufficient to cut it away 50 feet, and send spray 40 feet into the air. The shore at Belmar, on the contrary, is a sandy slope of say 1 on 6, and on this the waves would lose their force, so that light obstructions, such as brush, would cause the deposit of sand.

I saw the same action at Newport News. The embankment for the dry dock at that point extended 1 000 feet into the James River, and was exposed on both sides to the sweep of the waves. The material was light sand and loam, and the banks were cut rapidly by the waves until a slope of about 1 on 6 was reached, when all cutting ceased.

In conclusion, it is evident—

First.—That, whatever be the cause, there is a determinate tendency of inlets on this portion of the coast to elongate to the northward until they come to be practically parallel with the coast.

Second.—From the present action of the inlet at Shark River, and the experience with jetties at Manasquan, it is a mistake to run a jetty out straight and at right angles to the shore. The effect of this is to deflect the sand into the mouth of the inlet.

Third.—As the deep water in the inlet near its outer end is found on that side of the inlet towards which the shore currents set, it would seem that a tight jetty on that side (as I understand the author recommends) curving first concave to the shore so as to be nearly parallel to the shore at about the general range of high water, and from thence curving concave outward, so as to gradually deflect the currents oceanward, would be the most likely to insure deep water against its face. The fill would take place on the opposite side of the jetty. If a jetty should be placed on the opposite side of the inlet it should not be carried so far seaward, and should be only to prevent the ebb from cutting in the concave bend on that side. But it needs to be carried inland far enough to prevent cutting behind it, as has happened at Manasquan.

In the paper on the East Coast of England, referred to by the author, the harbor of Southwald is mentioned. The drift there is to the south, and two piers or jetties 300 feet long have been built in a southeast direction. These have caused a large mass of shingle to be heaped up to the south of the south pier. "There is also a large accumulation of beach and sand, within the entrance on the north side, formed of large bowlder shingle, which passes the pier-head and travels up the harbor. The north pier has recently been extended to a similar distance as the south pier, but the shingle from the northward is much in advance of it, and after passing the entrance collects around the south pier-head," and under certain conditions the harbor is choked.

It is clear that in this case the northerly jetty, that is the one first impinged upon by the shingle traveling along the coast, is a detriment.

It is very certain that vertical bulkheads are not good as a protection to banks exposed to the direct force of the waves, and much money is being wasted along our coasts in building them.

James B. Francis, Past President Am. Soc. C. E.—At the Seabright meeting, last year, I watched the operation of the waves coming from the south, and the movement of the sand was visible. The waves broke on the shore diagonally, every wave carrying northerly a certain amount of sand; when the force of the wave was exhausted the sand did not return, but was deposited, to be carried further north by the succeeding wave. There was, at the same time, a distinct northerly current, which I estimated at about 10 feet per minute, from the motion of a barrel about

100 feet from the shore floating parallel to it. The movement of the sand appeared to be quite independent of the current.

N. W. Eayrs, M. Am. Soc. C. E.—The chief claim to originality in the paper by Professor Haupt lies in the development of the "flood component" theory and the application of it to the improvement of harbors on sandy coasts. The general principles upon which such works of improvement must be based, viz., "1st, Keep out the littoral drift; 2d, let in the flood tide; 3d, conserve the ebb energy over a limited sector of the bar, and 4th, maintain a continuous reaction upon the ebb currents"—have been generally accepted by engineers, and, therefore, are (except in regard to the fourth principle, possibly) not new. But the establishment of the "flood component" as the cause of the littoral movement of material along the shores, and, consequently, the cause of the configuration of the harbors and inlets along our Atlantic coast would, it seems to me, reverse the order of the importance of the principles upon which harbor works should be based. This order Professor Haupt has very properly reversed, in the summary quoted above, making the effects of the flood tide the first consideration.

The littoral currents, and the transportation of drift material from point to point along the coast, have been made the subject of study by various investigators, and it has been generally assumed that the direction of the prevailing drift was dependent upon the wave movement along the shore, which movement was itself in the direction of the prevailing wind. This is an erroneous assumption, for numerous instances can be mentioned where the direction of the littoral drift is contrary to the direction of the prevailing wind. For example, along the north side of Long Island, N. Y., the drift is westward, while on the south shore it is eastward, and yet the direction of the prevalent wind is the same on both sides of the island. Again, on the south shores of Martha's Vineyard and Nantucket the prevailing movement of the drift is eastward, while the winds, at least during the portions of the year when they are of the greatest intensity, are generally from the north or northeast. This opposition between the direction of the littoral movements and the direction of the prevailing winds has been noted by many observers hitherto, but I think that Professor Haupt may justly claim the distinction of being the first to recognize the generality of the phenomenon, and to ascribe to it its true cause, namely, the modification of the ocean tide by the "nodal" points on the sea coast.

It is universally conceded that the most satisfactory remedy for the condition of harbors on sandy coasts is the application of the jetty system. This system consists, as is well known, in the construction of one or two (generally two) jetties, the object aimed at being primarily the concentration of the ebb current so as to produce the greatest scouring effect upon the bar, and only secondarily the protection of the harbor entrance from the littoral drift. In planning the location of two

jetties at the mouth of a harbor, the engineer is confronted by an indeterminate problem, to wit: How far apart must the sea end of the jetties be placed in order to give the requisite velocity to the issuing current and yet not impair the free access of the flood tide to the tidal basin. In the determination of the necessary velocity to be obtained in order to scour out the shoals the ordinary hydraulic formulas are not applicable; for they have been obtained from observations on rivers where the flow is constant in direction. But in a tidal channel the directions and strength of currents are hopelessly confused. Neither can any assistance be obtained from the usually accepted expressions for the velocity necessary to move sand or other material. I have, myself, obtained, with a current meter, bottom velocities in the entrance to Nantucket Harbor far in excess of that given as sufficient to transport material corresponding to that of the bed of the channel, and yet the bottom, which was plainly visible, was motionless.

The danger that may arise from placing the jetties so near together, in order to secure the necessary concentration of the ebb current, that the entrance of the flood tide is seriously impaired, is not so important in the case of a harbor which has a large fresh water drainage, as in the case of a strictly tidal harbor. In the former, the diminution of volume of the tidal prism may be made good by the fresh water discharge; in the latter, the efficiency of the ebb current depends upon the complete filling of the tidal basin.

It seems to me decidedly questionable whether works designed to confine and concentrate the ebb current can be so placed as to be no impediment to the entrance of the flood tide. I fully agree, therefore, with Professor Haupt's view of the greater probability of success in the improvement of a harbor which has no fresh water drainage, and where there is a "resultant movement of the drift" by the use of a single jetty properly placed than by two jetties. Certainly the first two conditions of success will be better fulfilled with one jetty than with two.

The improvement of Nantucket Harbor, Mass., is a case in point. This harbor has no fresh water emptying into it. The entrance is obstructed by a bar which had but 6 feet of water at low water in the channel up to a few years ago. The material composing this bar and the numerous shoals to the north on Nantucket Island undoubtedly come from the westward. After an exhaustive study of the physical characteristics of the bar, the late General G. K. Warren, Corps of Engineers, U. S. A., in 1879 proposed to attempt the improvement of the harbor by building a single jetty from the north shore of the island to the crest of the bar on the west side of the entrance. The jetty was commenced in 1881; in 1886, although the jetty had been built but about two-thirds the contemplated length, an increased depth of 18 inches had been obtained on the bar, and the areas included between the 9 and 12 foot contours

inside the bar had been materially increased. This improvement was the result of simply arresting the flow of sand from the westward, as there had been no concentration of the ebb current.

It would be unwise, however, to argue from the success which has so far been obtained at Nantucket, through the action of a single jetty, that a second jetty is unnecessary, for the directions which the littoral drift takes are somewhat peculiar. The prevailing easterly current, entering Nantucket Sound by Muskeget Channel, is laden with sand, which is deposited on the shoals between Tuckernuck and Great Point. A portion of this sand is washed along the north shore to the eastern extremity of Brant Point, where its further progress is stopped by the outflow from the inner basin. It is by the interception of this volume of drift material that the success which has attended the building of the west jetty has been achieved. Another portion of the sand current follows the outline of the bar until Great Point is reached, where it is deflected to the southward along Coatue Beach, carrying an enormous volume of sand until Coatue Point is reached on the east side of the harbor entrance. Here the velocity is checked by the outflow from the harbor, and the sand is deposited, forming Coatue Shoal.

That such is the movement of sand over the shoals is proved by the shape of Tuckernuck Shoal, and by the constant, though slow, eastward movement of the channel over the bar; and also by the large accretions to the shore of Coatue which have taken place on the northerly side of the spur jetties which were built for the protection of that beach.

Herein lies the necessity for a second jetty at Nantucket; a necessity which does not militate against the "flood component" theory, but is rather in favor of it.

In 1885 a board of engineer officers was convened to decide upon the location of a second jetty, which was deemed advisable, because, up to that time, the first, or west jetty, had produced no appreciable effect upon the bar. After a careful study of the harbor it was decided to place the jetty with its shore end resting on Coatue, and to continue it thence in a direction nearly parallel with the north shore of Brant Point, and about 2 000 feet distant therefrom, thence by a curve to the northward until a tangent to the curve would be parallel with the west jetty and about 1 000 feet distant from it. The inner portion of the second jetty, for about 2 500 feet from Coatue Beach, was to be built to a height of 5 feet above mean low water; the remainder of the jetty, about 4 500 feet in length, was to be submerged, the top at the height of mean low water. The principal object aimed at was to concentrate the ebb current over a limited section of the bar and produce the desired increase in depth by erosion.

In 1886, however, before the second or eastern jetty had been commenced, it was found that there was a good channel between Brant

Point and the outer slope of the bar, with 18 inches more water at mean low water than ever before, and in his report to the Chief of Engineers for the fiscal year ending June 30th, 1886, Colonel George H. Elliott, Corps of Engineers, then in charge of the work, reported as follows :

"Careful surveys were made in June last of the bottom of the entrance to the harbor to determine what effect, if any, the west jetty alone had produced on the bar, reports having been received that the water had deepened, and that vessels experienced much less difficulty last spring than formerly in crossing the bar at low tide. Our survey fully confirmed these reports.

"Although but little, if any, effect had been expected before the two jetties should have been extended until they should become parallel and contract the channel to the width contemplated, viz., 1 000 feet, it was found in the survey just referred to that on a line from Brant Point to the present end of the west jetty the least depth of water was nearly 1½ feet greater than at the time of our previous survey, and vessels had less difficulty in entering and leaving the harbor."

The construction of the second, or eastern, jetty was commenced in 1887, and to the present time a length of about 500 feet from Coatue is completed. I was informed, this summer, by the officers of the steamer running between Nantucket and Woods Holl, and by others who were familiar with the navigation of Nantucket Sound and Harbor, that the increased depth noted in 1886 is well maintained, and that vessels drawing 10 feet of water had passed in and out of the harbor without difficulty. As the mean rise of tide is but 3 feet, a draught of 9 feet hitherto practically shut out vessels even at high water.

Since the construction of the small portion of the eastern jetty as yet completed, the southern extremity of Coatue Flat, which formerly was bare at low water, has disappeared, and the shoal seems to be rapidly washing away.

This result of the work at Nantucket has, it will be seen, been obtained without any contraction of the ebb current and consequent erosion, but solely by arresting the flow of sand into the harbor. The rapidity with which Coatue Flat is disappearing, from the supply of sand necessary to its existence being cut off by the 500 feet of jetty so far built, shows how important that second or eastern jetty is to the improvement of the harbor.

The value of Professor Haupt's generalization lies in the application of it to works of harbor improvement, and the establishment of the "flood component" theory would necessarily modify, to some extent, the method of applying the jetty system to the improvement of harbors. It is to be hoped, therefore, that this paper will call out considerable comment.

L. M. Haupt, M. Am. Soc. C. E.—The author is gratified to note the substantial unanimity of opinion as to observed phenomena by the

distinguished members who have taken part in the discussion of this paper. This is the more remarkable since so little attention is given to the subject of harbor improvements by the civil engineers of the United States where the public works of this character are under the control of the Government Engineers. Yet it is a noteworthy fact that there has been no response from that source on the questions at issue. On the other hand, it is found that the proceedings of the British Institution of Civil Engineers are replete with protracted discussions on this subject which has done so much to extend the supremacy of the United Kingdom to the remotest quarters of the globe.

One of the most recent discussions by the institution took place at the meeting held February 4th, 1890, upon the comprehensive paper of Mr. William H. Wheeler, entitled "Bars at the Mouths of Tidal Estuaries," to which the author desires to call attention, and especially to his conclusions, reached after a careful review of the various types of estuaries in and near England. He sums up his remarks as follows:

1. That the causes which operate in maintaining the existence of bars in tidal estuaries come from the sea, and are not due to inland water.

2. That the dispersal of bars and the improvement of the outfall channels of estuaries can be accomplished by increasing the volume and velocity of the tidal water passing over the bar, and by giving a greater preponderance to the ebb current.

3. That those estuaries having natural deep water outlets should be taken as guides, and the lessons these teach are: (*a*) That the mouth of the outfall should be capacious enough to allow of the entry of sufficient tidal water to fill up all the estuary and upper reaches of the river; (*b*) that to insure a regular and undisturbed flow, the form of the estuary should be such as to afford a gradually decreasing width from the outfall (inward); (*c*) that the depth of water in the channel should be sufficient to allow of a free propagation of the tidal wave; (*d*) that the longer the run of tidal water that can be given the more effective will be the scouring action, and that therefore all obstructions to the free flow of the tidal current are prejudicial to the outfall; (*e*) that it is essential to secure adequate depth of the channel by natural scour or by dredging; (*f*) that training and fixing in one position the low-water channel in a sandy estuary is of first importance in securing the full advantage of the tidal scour and in maintaining a deep channel; (*g*) that water flowing in and out of an estuary which does not pass to or from the sea over the bar and by the main navigable channel is of no advantage to the outfall, and is better excluded, etc. * * *

4. That piers carried out from the shore are useless unless extended to deep water. That they should be so designed as, while not unduly throttling the inflow of the tidal water, they should effect the greatest amount of scouring force from the ebb current.

In closing his presentation Mr. Wheeler says also:

"As the chief object to be attained in the disposal of bars is the prevention of the material composing them traveling along the beach, from being carried into the mouth of the estuary, a single wall, carried up sufficiently high to arrest the progress of the sand or shingle, may, on some coasts, be sufficient. This wall would have to be placed on the side of the channel from which the littoral currents and prevailing gales

come; be carried out into deep water, and be concave in form, with the concave side next the channel. * * * The flood current, having to work along the convex side and around the projecting end, would keep it scoured. The ebb current from the estuary would always have a tendency to keep along the concave side and thus maintain deep water along the wall."

How completely these principles have been anticipated on this side of the Atlantic will be seen from the paper submitted to the Society, with its associated documents, and it is believed that an additional condition of great weight has been provided for by leaving a large aperture at the shore flank of the breakwater for the free admission of the flood tide, thus contributing greatly to the solution of what Mr. Eayrs characterizes, very properly, as the indeterminate problem of letting in the flood while concentrating the ebb tides.

The second jetty, which Mr. Eayrs states is a necessity at Nantucket, acts merely as a "grogive," and not as a "jetty." It would be quite as effective if placed a little farther east and made even shorter than in the proposed plan.

This is one of the peculiar cases where a reflex current exists, induced by the deeply indented bay on the north side of Nantucket, one horn of which (Long Point) intercepts the current and turns a portion of it south and west, forming Coatue Beach. The comparative charts of Long Point will show whether the parallel currents which maintain it are coincident or opposite, and southward, or the reverse.

Similar reversals of currents are found at Hatteras and Beaufort on the indented coast of North Carolina, where, in lee of the capes, there is an eastward drift current, and where the engineering features must be modified accordingly.

This coast furnishes another instance of sand moving in a direction opposite to the prevailing winds. In the Report of the Chief of Engineers for 1885 Colonel Elliott asserts that the eastwardly movement of the littoral drift is due to the prevailing westerly winds, yet he also acknowledges in the same report, the westerly drift of sand along Coatue Beach, or in a direction opposed to the wind, but no explanation is offered of this apparent anomaly.

The form of single breakwater which the author has designed for such difficult bars as obstruct the entrances to Charleston, Galveston, Aransas Pass, and other ports where there is a prevalent littoral drift, is best shown in the accompanying sketch of Aransas Pass, Texas (Plate XXXV), which is now in the hands of a private company for improvement. This pass has drifted more than 2 miles to the southward in the last forty years—or at the rate of 265 feet per annum. The author does not feel justified here in consuming the space necessary to explain the action and results to be expected from this reaction, fully believing that the chart, with the requirements of the problem, as already stated, will make the solution sufficiently intelligible.

As to the cause for the observed predominating direction of littoral drift, it may not be very material except as a scientific question, yet it was the investigation of this question that led to the practical deductions already submitted which were so emphatically indorsed by the American Philosophical Society in December, 1887.

Mr. Collingwood seems to lean to the wind wave theory rather than to that of the "flood component," and his views appear to be based upon the opinion of a single Signal Service keeper at Shark River. It seems to the writer that a cause such as the latter ascribes to the northward movement of the inlet at Shark River should apply with at least equal force at Barnegat or Absecon Inlets some miles further south and exposed to the full fetch of the Atlantic's waves, but where the littoral movements are in the opposite direction, or southward.

The effects of the storm of September 12th, 1889, which veered from north to northeast, are noted by Mr. Collingwood, where he says: "The waves would break on the shore of the spit and carry great quantities of sand northwesterly into the river. On the contrary, the northeast gale drifted large amounts of sand southwesterly so as to cover fences in the vicinity."

The writer understands this to mean that while the sand on shore was driven before the wind, that under water moved at right angles thereto; a physical impossibility, unless another force were brought into action, and that the "flood component" or northwardly moving littoral current. If it were wind alone it would seem that the northern beach should have been driven southward, and the author made a visit to this inlet as soon after the storms as possible to determine this question, and found that there had been no recession of the mouth to the southward. In his conclusions Mr. Collingwood does not lay stress on the cause, but emphasizes the facts which it was the intention of the author to draw out, viz.: that at most inlets there is a predominating direction of movement which furnishes the key to the successful design of works for the improvement of the bars obstructing the entrance from the sea. How closely he accords with the writer will be seen by comparing his second and third deductions with the plans for Aransas Pass. In reference to the harbor of Southwald, the defect arose from a violation of the conditions in not extending the piers to deep water, nor were they of proper form nor in proper position. The angle to the south made a "lee" in which was caught the shingle drifting past the pier-heads in comparatively shallow water. It is not surprising that "the harbor is choked."

There are many interesting facts in this discussion, especially as to the rapid rate of advance of the Shark River inlet and of the new cut, for which the author feels personally indebted to his colleagues who have aided in advancing a knowledge of the subject. It is hoped that more extended interest may be manifested in the development of our commercial facilities by the Society generally.

Mr. H. C. Ripley, C. E.—I have read the paper of Professor Haupt on "Littoral Movements of the New Jersey Coast, with remarks on Beach Protection and Jetty Reaction," with much interest. This interest is intensified in no small degree by the fact that my own experience has led me to the same conclusions with regard to many of the phenomena treated so ably by the author. Want of time will not permit me to enter exhaustively into the discussion of this paper. There are two features of it, however, that are of such paramount importance in the solution of the problem of harbor improvements that I would be glad to add the testimony of my own observations and studies. These features are the "Resultant Direction of Drift" and "Reaction *vs.* Velocity."

With regard to the former it may be said that the history of Manasquan River Inlet is almost identical with that of many of the Gulf inlets, particularly that of Aransas and Galveston, although, perhaps, the cycle of changes has not been so completely observed at these latter places as at the former.

At Aransas the southerly movement of the pass is well authenticated. An examination of the topography of Mustang Island, south of the present entrance, shows unmistakable evidence of the pass having been at some more or less remote period about 1 700 feet further to the southwest than its present position; when, by some extraordinary storm effect, the channel undoubtedly broke through St. Joseph Island at the foot of Aransas Bay to repeat again its southerly movement. (See chart opp. p. 1470. Report of C. of E. for 1882, part 2.)

The history of the changes in this pass not only shows the resultant direction of drift but also the cyclic character of those changes and the danger which may be apprehended and must be guarded against in any plan for its improvement. For while works are being constructed at the entrance of the pass to deepen the water there, a new channel is liable to break through the island further to the north, if not guarded against, which, with its shorter distance to the Gulf, and consequently greater water slope, will baffle every effort toward closing it, and hence in such an event the works at the old entrance would have to be abandoned. At Galveston the pass has not moved in so marked a degree. Yet the main channel across the bar has suffered changes in position which indicate with a great degree of certainty the resulting direction of drift, and also the cyclic character of those changes. In 1851 the position of the main ship channel was southeast from the light vessel, in 1867 it was south-southeast, and in 1872 it was directly south. A storm in 1875 caused a channel, which at once became the main ship channel, to break through the bar at a point directly east of the light vessel, in the prolongation of Bolivar Channel, from whence it commenced again its southerly movement. In 1880 we find its axis about one mile further to the south, and had it not been for the construction of the south jetty, there can be no doubt but that it would have repeated its previous history and gradually

worked its way to the extreme southerly position it occupied in 1872. Could there be any more certain indication of the resultant direction of drift than is shown by the history of these movements?

The effect of reaction due to a curved form of jetty will next be considered. Some years ago (1878), in connection with a survey of Brazos River, Texas, the writer made a number of careful cross-sections of the stream at distances apart of about one mile. These cross-sections were eighteen in number, and a comparison of them gave the following results:

First.—The average width of the river was 7½ per cent. less in bends than in reaches.

Second.—The average sectional area was 13½ per cent. greater in bends than in reaches.

Third.—The average maximum depth was 58 per cent. greater in bends than in reaches.

Fourth.—The average hydraulic radius was 24½ per cent. greater in bends than in reaches.

Fifth.—The average ratio of the maximum depth to the hydraulic radius was 29 per cent. greater in bends than in reaches.

Sixth.—The average wetted perimeter was 6½ per cent. less in bends than in reaches.

It will thus be noted that while the average width and wetted perimeter are slightly less, the sectional area is greater and the maximum depth is decidedly greater in bends than in reaches. It thus becomes very apparent what an immense advantage the reaction principle possesses in deepening channels, and it would seem to be beyond question that the same principle applied to the form of a jetty or wall, where the water is constrained to flow along the concave face of such a structure, would produce results greatly in excess of what could be produced by a right line structure.

In conformity with this principle, the writer, in 1883, designed a curved jetty, in place of the proposed straight north jetty, in connection with the improvement of Galveston Harbor. This form and location of the north jetty gave such promise of immediate and sufficient results, that the U. S. Engineer officer then in charge of the work proposed its adoption, in his annual report of 1884. (See Report of C. of E. for 1884, part 2, p. 1298; also chart opp. p. 13.)

The failure of the appropriation, however, prevented its construction, and hence was lost to science this opportunity of demonstrating practically the truth of one of the most important principles in the problem of the improvement of harbors on sandy coasts.

Mr. Haupt.—The author desires to add, in response to the very interesting contribution from Mr. H. C. Ripley, that he was not aware until reading this paper that the concave, detached breakwater, dotted in plan on the charts of Galveston Harbor for 1884 and called there "Col. Mansfield's Jetty," was due to Mr. Ripley, the able civil assistant engi-

neer who has been connected with this work for about eighteen years. This jetty the writer has frequently declared to be the best part of the design, yet requiring a few modifications to increase its efficiency. Had it been constructed first the $2 000 000 already spent on the south jetty without beneficial effect* might have been saved. The correctness of the reaction principle, however, does not fall from lack of this instance, for there are other concave jetties to verify its truth, notably at Swinemunde and other Baltic ports, and at Bilbao, Spain, but our Government has thus far failed to profit by them.

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* *Vide* Ex. Doc., H. R. 454, 51st Congress, page 15 *et seq.*

PLATE XXIII.

VOL XXIII P154.

PANORAMIC VIEW OF MANASQUAN INLET SHOWING REMAINS OF JETTIES
LOOKING LANDWARD FROM THE SOUTH SPIT.

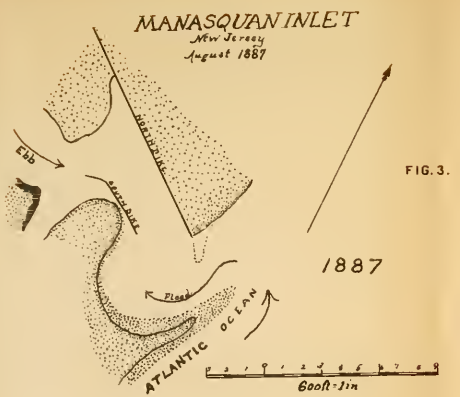


FIG. 3.

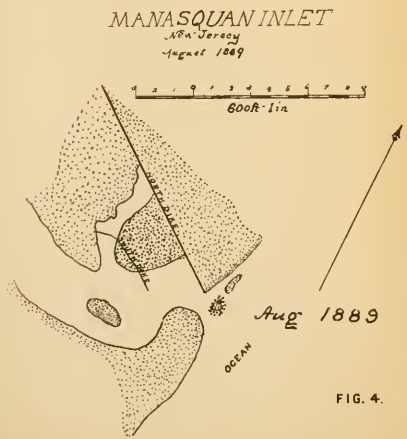


FIG. 4.

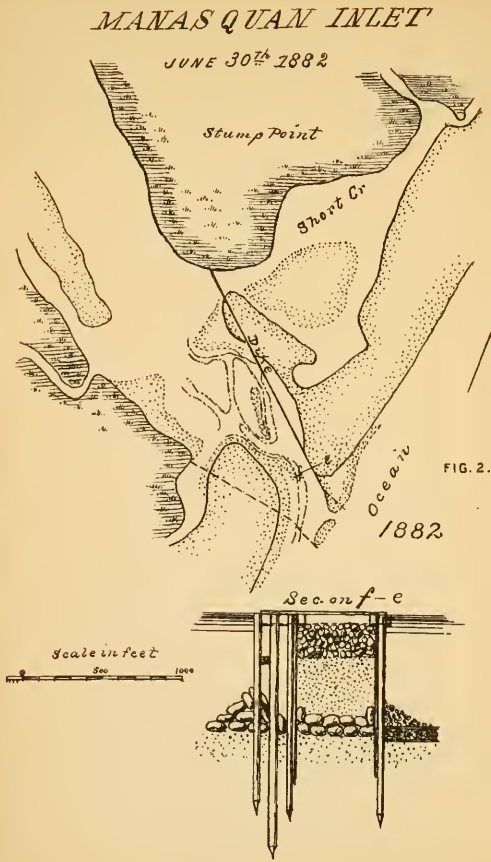


FIG. 2.

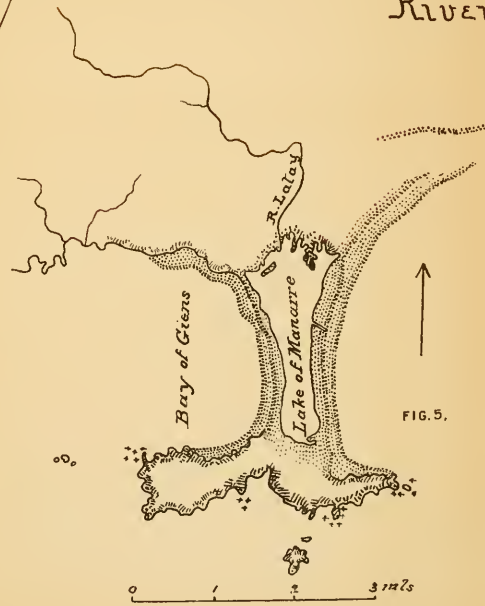


FIG. 5.

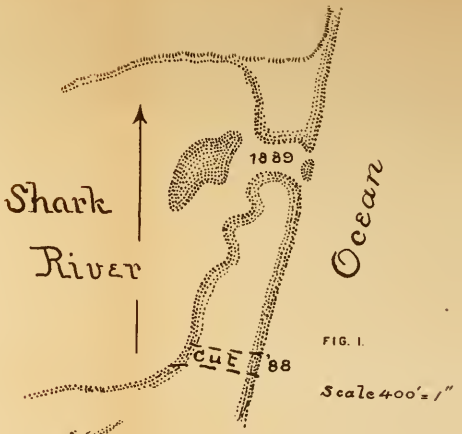


FIG. 1.

PLATE XXIV.
TRANS. AM. SOC. CIV ENGRS
VOL. XXIII No 451
HAUPT ON
LITTORAL MOVEMENTS

PLATE XXV.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXIII. No. 451.
HAUPT ON
LITTORAL MOVEMENTS.

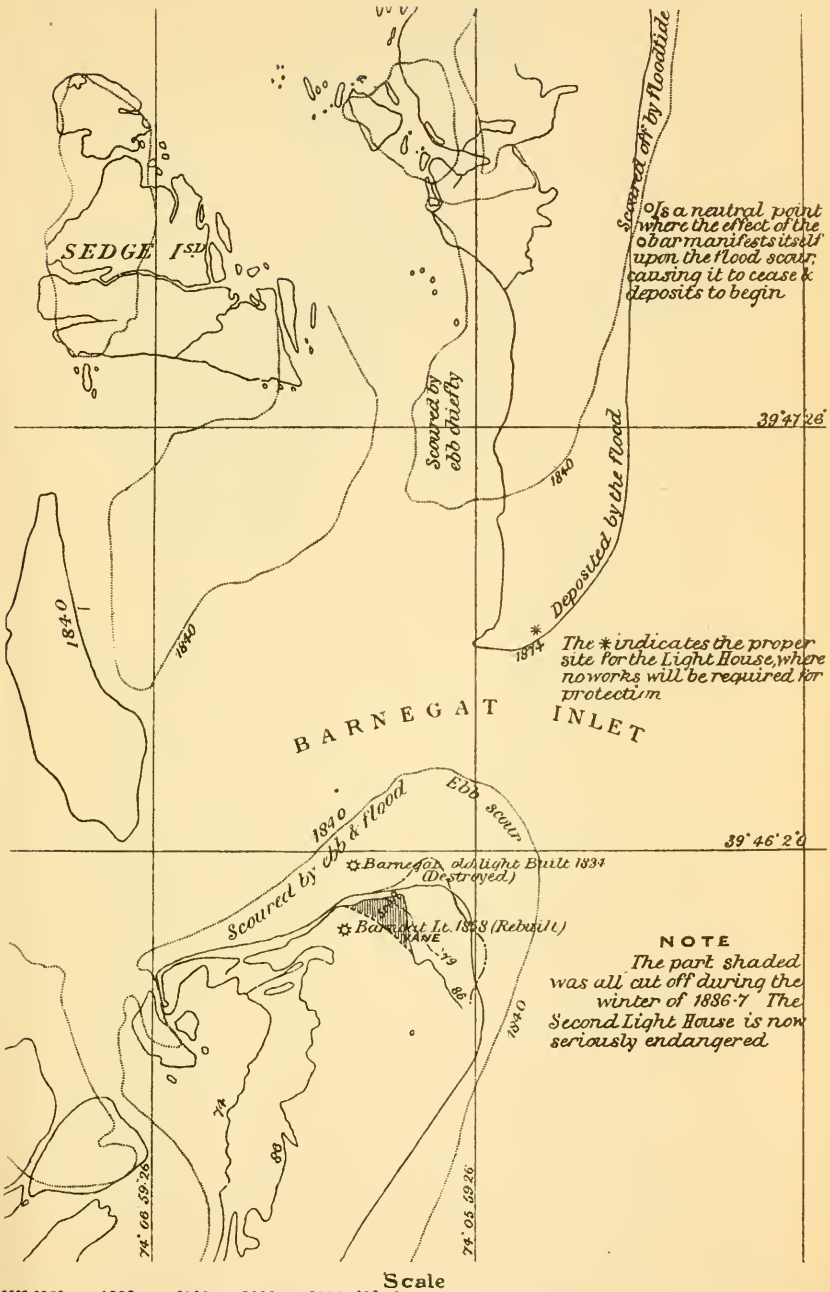


PLATE XXVI
TRANS AM SOC CIV ENGRS
VOL XXIII Nº 451
HAUPT ON
LITTORAL MOVEMENTS

1574

LITTLE EGG
Little
Parker
Parker
Isle Mordecai
Gr
Johns Sedge
Bay
Gr
Gr
Gr

PLATE XXXII
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII. No 451.
HAUPT ON
LITTORAL MOVEMENTS.

COAST CHART No. 109
BOSTON BAY
AND APPROACHES
Triangulation by C. O. BOUTELLE and Capt T. J. CRAM U.S.A. Assts
Topography by H. L. WHITING, J. R. GLUCK, A. M. HARRISON and B. M. HACHE Assts
Hydrography by the parties under the command of
Lieuts Comdg C. H. DAVIS, C. E. M. BLAIR, H. S. STELLWAGEN
C. H. P. RODGERS and W. G. TEMPLE U.S.N. Assts
Scale Fifteen
LIGHT HOUSES
VARIATIONS OF THE MAGNETIC NEEDLE
LIFE SAVING STATIONS
ABBREVIATIONS OF BOTTOMS
SOUNDINGS
SAILING DIRECTIONS
TIDES
NOTE
PLATE XXXI
TRANS AM SOC CIV ENGRS
VOL XXIII No 451
HAUPT ON
LITTORAL MOVEMENTS
SALEM
ROXBURY
QUINCY
Scale of Yards
Scale of Miles
70° 55'
70° 50'
70° 45'
70° 40'
70° 35'

PLATE XXXIV.
TRANS. AM. SOC. CIV ENGRS.
VOL. XXIII. No 451.
HAUPT ON
LITTORAL MOVEMENTS.

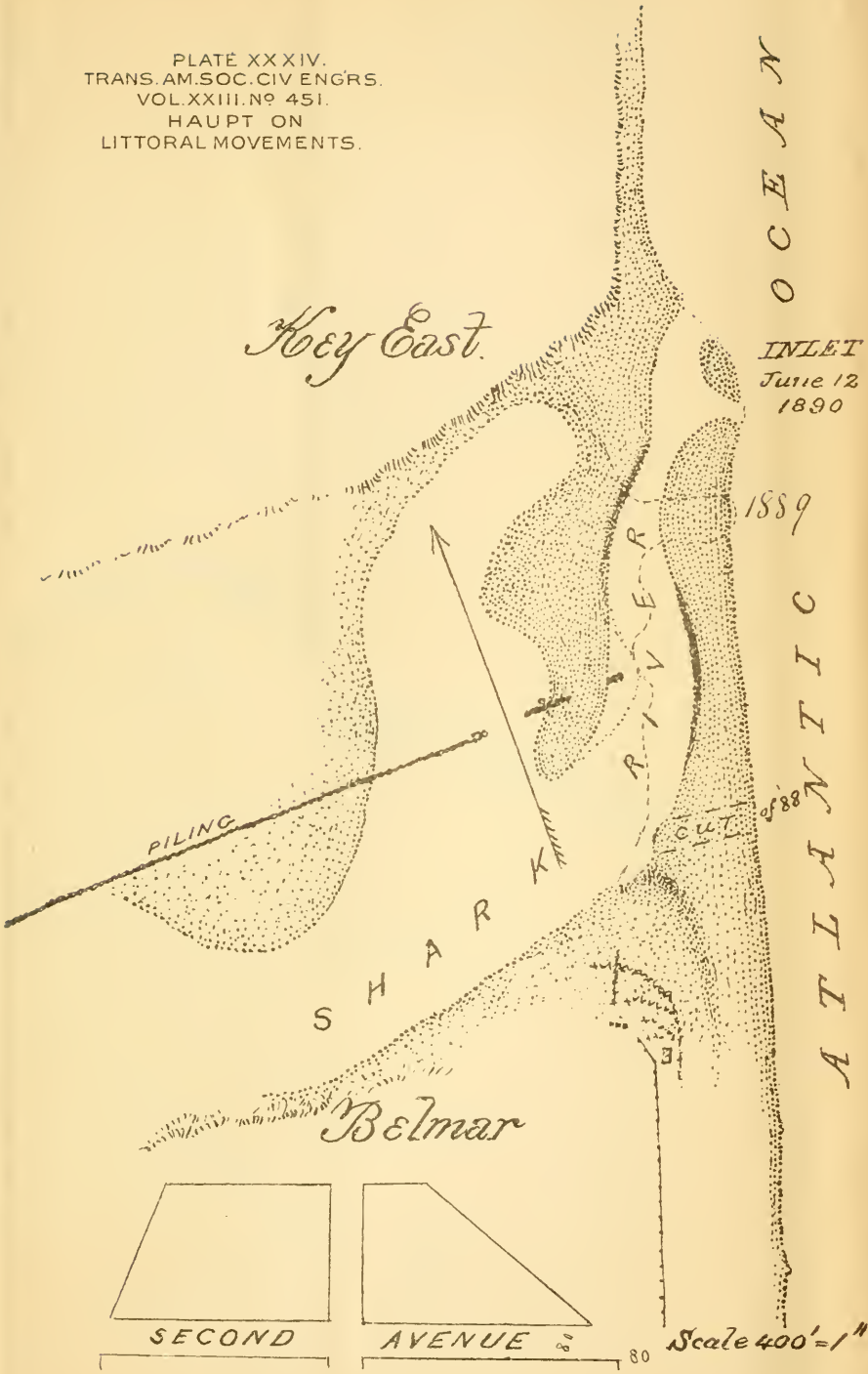
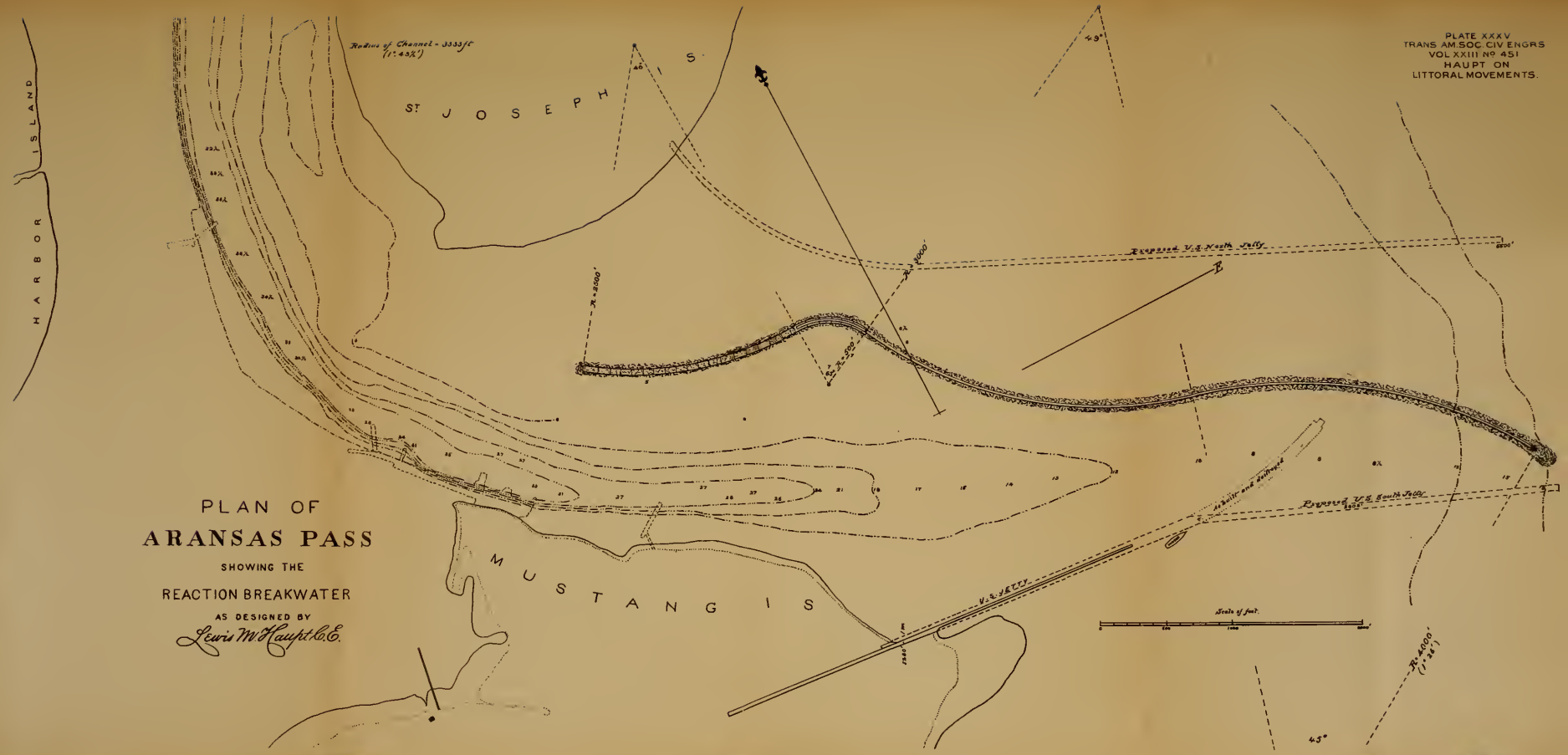


PLATE XXXV
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII No. 451
HAUPT ON
LITTORAL MOVEMENTS.



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TRANSACTIONS.

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452.

(Vol. XXIII.—October, 1890.)

ON THE DESIGNING AND ERECTION OF THE OAKLEY ARCH.*

A FULL CENTERED OBLIQUE CONSTRUCTION OF EXTREME SKEW, TO CARRY A RAILWAY EMBANKMENT OVER ANOTHER DOUBLE TRACK RAILWAY.

By J. FOSTER CROWELL, M. Am. Soc. C. E.

WITH DISCUSSION.

This paper has been written in compliance with requests from members of this Society, who have from time to time expressed to the author a desire to have placed in permanent form an exposition of the methods pursued in its design and execution.

The structure is notable for its obliquity of 42 degrees 30 minutes, which is about the most unsafe angle to which a semi-circular arch can be applied, security being attained by an original device, and also as an example of excellent mechanical execution and workmanship which are attested by the series of photographs taken during progress and here reproduced. (Plates XXXVI to XLIII.)

* On the line of the Cincinnati and Richmond Railroad (Pennsylvania System) at Oakley, 9 miles from Cincinnati, at its crossing under the Baltimore and Ohio Southwestern. Built in 1887.

The peculiar difficulties of erection, due to situation, treacherous natural formation and necessity of maintaining overhead without interruption the heavy traffic of a rival railroad, presented conditions somewhat unusual and complicated, and it may, perhaps, be interesting to follow briefly the different stages (illustrated by the progress photographs) before taking up the consideration of the methods employed in designing the structure.

Situation.

The Cincinnati, Washington and Baltimore Railroad (now the Baltimore and Ohio Southwestern) crosses the Duck Creek Valley, near Oakley, Ohio, on an embankment from 30 to 40 feet in height. About 300 feet from the western end of the embankment is a stone arch culvert 30 feet in diameter, providing passage for Duck Creek. The Cincinnati and Richmond Railroad was located on the west side of the creek, to cross diagonally, at a grade about 20 feet lower than the Cincinnati, Washington and Baltimore, as it stood at that time, between the culvert and the end of embankment. On the south side of the embankment the crossing did not extend quite to its end, but on the north side, because of its angular direction, it was contiguous to the natural surface, and cut into the latter a few feet beyond. (See Plate XXXVI.)

Just at the crossing and for a total distance of, perhaps, 100 feet, the grade of the Cincinnati, Washington and Baltimore was level, being a depression with a 1 per cent. gradient of considerable length rising in each direction. It was desirable to raise the Cincinnati, Washington and Baltimore grade over the crossing to provide sufficient head-room for the Cincinnati and Richmond, and it was agreed between the two companies that, as it was contemplated by the former to raise the grade 11 feet still higher eventually, so as to take out the dip, the new structure should be constructed of sufficient dimensions to support a double track embankment of the required future height, although the present embankment is of single track width.

Apart from other important considerations, economy called for a masonry arch, and after thorough comparison of the relative cost and expediency of various forms of construction (see note at end of this article), a semi-circular skew-arch was determined on, the barrel of brick, foundation-walls, bench-walls, spandrels, parapet, quoins and backing of stone.

Owing to the arrangement of grades on the Cincinnati, Washington and Baltimore, before referred to, it had been the practice to pass trains of all classes over this spot at high speed in both directions to acquire momentum for the opposing grades, as train-men say, "making the run for the hill." It, of course, would not be permissible to continue to do this during the progress of the work; but, on the other hand, speed could not be reduced as much as was strictly desirable, without causing considerable delay in getting under way again on the grade, and as the constructing company was liable for all damages and expense, and the other company on guard against any avoidable interference with its traffic, it was necessary to adopt a system of temporary works that would safely permit of a fair speed, and at the same time cause as little hinderance as possible to the prosecution of the work. Furthermore, it must be of such character that it would easily permit of the raising of grade during the progress of the work to the extent necessary to clear the crown of the arch. On account of the proximity of the 30-foot arch culvert, before referred to, on one hand, and of a deep cut beginning at the western end of the embankment, on the other, it was not possible to make a complete shift of the Cincinnati, Washington and Baltimore track, so as to clear the projected work, except at unwarrantable expense, and with the result of still further interfering with the train movement; it was practicable, however, to swing the track a short distance to the north, and this was availed of to adopt the alternate-trestle system, which was so successfully used throughout the work, that although the Cincinnati, Washington and Baltimore management had its own watchmen employed night and day ready to detect and report any signs of short-comings, there were no cases of delayed trains, no complaints of any character, and no accidents.

It is proper to state in this connection that while the Cincinnati, Washington and Baltimore people were so constantly on the alert, as it was their duty to be, they invariably extended the utmost courtesy and co-operation, and the writer takes pleasure in this opportunity to acknowledge especially the kindly offices of Mr. C. W. Stone, Master of Road of that Company, who was most directly concerned.

To properly handle the freight trains at the reduced speed a helping engine was used, and this expense, borne by the constructing company, was a formidable item, not due however to the manner of construction, but resulting from the unavoidable conditions of grade.

The length of Cincinnati, Washington and Baltimore track to be temporarily supported was 200 feet, and as the entire foundation of the arch was to be on piling it would not have been possible to make use of trusses for temporary works.

The first trestle was on the center line of the main track; the alternate trestle was parallel to it on the north, and as far from it as the permissible swing of the main track made possible; a switch at the west end of the swing enabled each trestle to be used in turn as a spur to unload material.

The first step was the building of the first trestle by driving with a track pile-driver three lines of piles directly into the embankment, one line in the center of track and the others beyond the ends of the ties and as far out as the machine would reach, the bents being spaced 16 feet. Owing to the limited height of the guides piles of sufficient length to reach the bottom of proposed excavation could not be used, but by starting them in holes dug for the purpose and using a follower they were sent as deep as possible and penetrated the natural surface beneath the embankment. They were cut off about 3 feet below base of rail. Caps 12 x 12 inches x 14 feet long were then drift-bolted to the piles at right angles to the track, cross trenches being cut to admit them, and the shoulders of the bank cut away so as to allow double stringers or balks, each 10 x 20 inches x 32 feet long, to be placed on the caps just outside of the ties, but without being secured. The stringers were made of this unusual size to guard against yielding in case of some pile being undermined or developing a defective bearing during the removal of the embankment. They were arranged to break joint and were bolted together. They were next slipped under the track-ties, one side at a time, and secured to the caps. Finally the track-ties were replaced with bridge-ties secured with guard timbers, and, the track being now on the trestle, the work of excavation and removal of the embankment in benches proceeded. Diagonal and horizontal bracing was spiked to the piles as they were gradually uncovered, and lagging was inserted at the end of each bench to hold up the exposed faces of the bank (see Plate XXXVI). After sufficient of the bank had been removed the excavations for the outlying parts of the foundations were begun on both ends, and the bearing-piles driven for the structure. This work was carried as near to the track-trestle as was deemed safe on both sides, leaving a gap to be completed later. Besides the bearing-piles, retaining-piles were driven

on the back lines, and left high to aid in holding up the bank. The bearing-piles were cut off 2 feet below water level and capped with 12 x 12-inch grillage in two directions, halved together and drift-bolted. The spaces between piles were cut out and filled with strong concrete, well rammed and carried up flush and full to the upper surface of grillage. Simultaneously the piles for the alternate trestle were driven and the caps and stringers laid. The stone footings, laid in courses, were begun as fast as the grillage was ready to receive them. When all the foundations that could be reached without disturbing the track trestle had been carried up to the ground level, the track was swung over to the alternate trestle, the first trestle was removed and the gaps previously left in the foundations were filled. The alternate trestle continued to be used until the arch was turned for a sufficient length to carry the main track, which was thereupon shifted back and the trestle finally removed. The centering (see Plates XXXVIII, XXXIX and XL) was designed to carry the arch and its load, and was not struck until some time after the completion of the entire work.

As the bench walls were carried up, the adjacent parts of the trestle were supported upon them by "A" frames and braces, which were cut off at each successive course, and the embankment was back-filled and rammed in layers, so that while the arch was being turned there was only a comparatively short length of trestle in service.

Great trouble was experienced, as was to be expected, from the very faulty natural formation, which caused frequent slips and continuous interruptions to the work on the foundations. The bed of the creek valley was underlaid with quick-sand on hard blue clay. Above the sand was about 10 feet of alluvial deposit extending back to the hill-side, into the foot of which the west foundation extended. The hill-side was formed of alternate layers of blue clay and dry sand; the clay being very hard and difficult to work when dry, and very slippery and treacherous when wet. The difficulty was of course increased by the weight of the superincumbent embankment formed of similar material, and by its constant tendency to slide. A remarkably wet season added to these causes, resulting in formidable delays and expense.

The direct supervision of the work and the carrying out of the plans were in charge of Mr. M. P. Paret, M. Am. Soc. C. E., Division Engineer of the Cincinnati and Richmond Company, to whom and his assistants credit is due for successful accomplishment.

The Methods of Design.

The practical methods adopted by the author in designing this arch are valuable because of—

a. Their simplicity and conciseness.

b. Their dependence upon computations and not upon intricate and difficult drawings.

c. Their reliability and interchecking character. Drawings are desirable in amplification for explanatory purposes, for convenience of reference and for record, and reproductions of the full set used in this case are here given (Plates XLIV to L), but they are simply diagrams, and were constructed from the computed quantities given in the Tables (1 to 4), which were calculated and verified in advance. The drawings were not used by the mechanics, full sized templates readily laid down from the tables being furnished them by the engineers.

The methods are presented here without theoretical demonstration excepting in the general way made necessary for their elucidation, and the results given in the different tables are the actual figures used in the case under discussion. The necessary working formulas are stated and equations, reduced to their simplest form, are subjoined to the tables. Those who desire the theory are referred to the very valuable "Essay on Oblique Bridges, by George Watson Buck, M. Inst. C. E., with addenda by W. H. Barlow, M. Inst. C. E.," to which source the writer is indebted for most, though not all the steps followed, he having, previous to his knowledge of the existence of the above work, designed and erected a skew arch* of 50 feet span with heliocoidal joints carrying a double track railway embankment, in which similar practice was followed; but he none the less desires to acknowledge gratefully his obligation to Mr. Buck for the very simple and lucid demonstrations which distinguish that essay.

Table No. I is made up of data and derived dimensions. There are five governing quantities which affect the skew construction.

The rectangular diameter of the intrados at the springing line.
The radius of the intrados.
The depth of voussoirs.
The angle of obliquity, being the acute angle between face and axis.
The rectangular width of the structure between faces.

---

* See "The Behavior of two Brick Arches," by J. Foster Crowell, Proceedings of Engineer's Club of Philadelphia, Vol. V, No. 3, November 21st, 1885.

As we shall see, all other dimensions and values, both actual and developed, are derived from these by the use of the trigonometrical functions and the equation of the ellipse.

TABLE No. 1.

Oakley Arch.		Data and Derived Dimensions.	
Θ	Angle of obliquity.....	.....	42° 30′
r	Radius of cylinder.....	.....	14. Feet.
e	Thickness of voussoirs....	.....	2.167′
b	External width.....	.....	54.167′
$A\ B$	Direct span.....	.....	28.0
$A\ C$	Oblique span.....	$= 2\ r$ co sec. $\Theta = 28. \times 1.480187$.....	41.44
$B\ C$	Obliquity of arch.....	$= 2\ r$ cot. $\Theta = 28 \times 1.091310$.....	30.5567
$A\ F$	Length of arc.....	$= \pi\ r \quad = 14 \times 3.14159$.....	43.9823
$M\ C\ I <$	Intradosal angle.....	$\tan.\ M\ C\ I = \frac{\text{Cot. } \Theta}{\frac{1}{2}\pi} = \frac{1.09131}{1.5708} = .69475 = \tan.\ \beta$..	34°47′22″
ϕ	Extradosal angle.....	$= \frac{r+e}{r} \tan.\ \beta$.....	38°43′
δ	Twisting rule angle.....	$= \phi - \beta\ (= 38°\ 43' - 34°\ 47'\ 22'')$.....	3°55′38″
$A\ C'$	Length of heading spiral..	$= \pi$ r., sec. $\beta = 43.982 \times 1.217561$.....	53.5499
	Number of voussoirs.....	width of each voussoir $= \frac{53.5499}{39} = 1.373$.....	39.
$C\ D$	Length of impost.....	$= b$ co sec. $\Theta = 54.167 \times 1.480187$.....	80.177
K	Divergence of courses,.....	$= b$ co sec. Θ sin. $\beta = 80.177 \times .5704$.....	45.733
	Number of voussoirs.....	intersected by spiral off impost	34.
	Number of courses.....	of brick to each voussoir.....	6.
	Theoretic eccentricity.....	$= \frac{\text{Cot.}^2\ \Theta}{\frac{1}{2}\pi}\ (r+e) = \frac{1.09131^2}{1.5708} \times 16.167$.....	12.27
	Also.....	$= 14 \times \text{Cot. } \Theta \tan.\ \phi = 14 \times 1.09131 \times .801629$....	12.27

The designating letters in the table correspond to Figs. 1 and 2, Plate LI, a description of which is necessary before proceeding further.

Fig. 1 is a plan and rectangular section of the intrados, showing the limits of the skew. The inner surface of the semi-cylinder is developed in the horizontal plane of the springing line without reversion, it being assumed that the surface is transparent so that lines on its under side are visible in their true relation from above. The parallel lines at right angles to the lines joining the developed terminations of each face are the developments of helices of the cylinder. In the development they are right lines making with the sides of the cylinder a common angle called "the angle of the intrados."

The faces of the intrados, which are bounded by vertical planes in their true position, are in their development reverse curves, the exact determination of which will be given further on, under the head of "Soffit Development." It is manifest that the other boundaries of the development are right lines in the same plane. All lines which in the development make the intradosal angle with the springing line

become helices when restored to their true position. Such of the helices as intersect the face at joint points are the lines of spiral joints in the intrados.

Fig. 2 is a plan and section of the extrados with the development turned back and reversed, so that it lies with its underside uppermost; this being done to avoid obscuring other parts of the plan. No practical use is here made of the extradosal development, it being introduced simply for illustration. The angle ϕ, or the "angle of the extrados," is required in the computation, but is easily obtained trigonometrically without development.

Fig. 2 also shows an elliptical section of the intradosal and extradosal surfaces revolved into the plane of the springing line. Both of these curves are true ellipsi and require to be accurately computed, so that any portions of them can be laid out with refinement in full size as may be required.

Table No. 2 gives the deduced values of axial dimensions of the intradosal and extradosal ellipsi, their equations and rectangular coordinates, of which Plate XLVI is the working record. Upon the

TABLE No. 2.

OAKLEY ARCH. ABCISSAS AND ORDINATES FOR FACE ELLIPSI.

Equation for Intradosal Ordinates $y = \frac{b}{a}\sqrt{a^2 - x^2} = \left(\frac{14}{20.72} = .6757\right)\sqrt{20.72^2 - x^2}$

" " Extradosal " $y_1 = \frac{b_1}{a_1}\sqrt{a_1{}^2 - x_1{}^2} = \frac{16.167}{23.93}\sqrt{23.93^2 - x_1{}^2}$

Note: Values of x_1 are same as values of x within limits of x.

Values of x and x_1	Values of y.		Values of y_1		Values of x and x_1	Values of y.		Values of y_1	
Feet.	Feet.	Inches.	Feet.	Inches.	Feet.	Feet.	Inches.	Feet.	Inches.
0	14		16	2	16	8	$10\frac{3}{4}$	12	$0\frac{1}{2}$
1	13	$11\frac{3}{4}$	16	$1\frac{3}{4}$	17	8	$0\frac{1}{16}$	11	$4\frac{1}{2}$
2	13	$11\frac{1}{4}$	16	$1\frac{1}{4}$	18	6	$11\frac{1}{8}$	10	$7\frac{3}{4}$
3	13	$10\frac{5}{16}$	16	$0\frac{3}{8}$	19	5	7	9	$10\frac{1}{2}$
4	13	$8\frac{7}{8}$	15	$11\frac{3}{8}$	19.5	4	$8\frac{3}{4}$	9	$4\frac{1}{2}$
5	13	$7\frac{1}{16}$	15	$9\frac{3}{8}$	20	3	$7\frac{7}{8}$	8	$10\frac{3}{8}$
6	13	5	15	$7\frac{1}{4}$	20.5	2	$0\frac{1}{2}$	8	4
7	13	$2\frac{1}{8}$	15	$5\frac{5}{8}$	20·72	0	0		
8	12	11	15	$2\frac{3}{4}$	21			7	$8\frac{1}{4}$
9	12	$7\frac{3}{8}$	14	$11\frac{3}{4}$	21.5			7	$1\frac{1}{2}$
10	12	$3\frac{1}{8}$	14	$8\frac{5}{8}$	22.5			6	$4\frac{3}{8}$
11	11	$10\frac{3}{8}$	14	$4\frac{1}{4}$	22			5	6
12	11	$4\frac{7}{8}$	13	$11\frac{3}{4}$	23			4	$5\frac{3}{8}$
13	10	$10\frac{7}{8}$	13	$6\frac{3}{4}$	23.5			3	$0\frac{3}{8}$
14	10	$3\frac{7}{8}$	13	$1\frac{3}{8}$	23.75			2	0
15	9	$7\frac{7}{8}$	12	$7\frac{1}{4}$	23.93			0	0

Abcissas are measured from center. See record plan, Plate XLVI.
Ordinates " vertically from horizontal axis.

proper delineation of these curves much of the exactitude of the ensuing work depends. The next step is to lay out the face joints, but before doing so, it will be necessary to compute the curves of the soffit development and to determine the number of ring-stones.

The Soffit Development.

Fig. 3, Plate LI, represents the development of the intersection of one face with the soffit, the two halves of the curve being symmetrical, and, of course, applicable to both faces. One set of ordinates, therefore, answers for each of the four quadrants. It now becomes necessary to show how this development is to be obtained without projecting it. Let AaB, Fig. 4, be the half of a semicircular arch, the obliquity of which is BDC, and suppose it is required to produce the development DeE by means of ordinates obtained by calculation.

Let the arc AB be divided into a convenient number of parts and its development, BE, into the same number. Suppose a to be one of the divisions of the arc and b its corresponding division in the development, such that $Eb = Aa$.

Also let

AC, the radius	$= r$.	$Ce = r \sin. \varepsilon$	
$\angle ACa$	$= \varepsilon$	$cd = r \sin. \varepsilon \cot. \Theta = be$	
$\angle CDB$	$= \Theta$	$Eb \times \tan. \beta =$	bf
$\angle BED$	$= \beta$	$be - bf \quad =$	fe

Thus having found a sufficient number of distances fe, corresponding to divisions of the arc AB or to its development BE, and consequently to DE also, let DE be divided into the same number of parts, as shown in Fig. 3. (Note.—In practice it is not necessary or desirable to make the parts equal throughout the whole arc, for that would result either in an insufficient number of ordinates at the sharpest part of the curve, or an unnecessary number at the flat parts. It is therefore more convenient to arrange them in several groups, each group consisting of equal parts, with corresponding groups on the development.)

At each of the divisions draw the ordinates fe, Fig. 3, making all the angles, Dfe, each equal to the complement of BED, which is the intradosal angle before treated of, and upon these ordinates set off the distances fe, as previously calculated; then the curve DeE, drawn through the points thus obtained, will be the development required.

Only one-half the development is shown, because the ordinates for the second half are equal to those on the first half, but applied on the contrary side of the line, DE.

TABLE No. 3.

Oakley Arch. Soffit Development: Face of Arch.

I. Angles ACa or ϵ	II. Log. Sin. ϵ	III. Log. r cot. Θ Sin. ϵ or Log. be.	IV. Nat. num. be.	V. Values of bf.	VI. Ordinates e or $be—bf$.	VII. Abcissas on Heading Course.
° ′						
4 30	8.894643	.078719	1.199	.7639	.435	1.339
9	9.194332	.378408	2.390	1.5278	.862	2.678
13 30	.368185	.552261	3.566	2.2917	1.274	4.017
18	.489982	.674058	4.721	3.055	1.666	5.355
22 30	.582840	.766910	5.847	3.819	2.028	6.694
27	.657047	.841123	6.936	4.583	2.353	8.033
30	9.698970	.883046	7.639	5.092	2.547	8.925
33	.736109	.920185	8.321	5.601	2.720	9.817
36	.769219	.953295	8.980	6.110	2.870	10.710
39	.798872	.982948	9.615	6.619	2.996	11.602
42	.825511	1.009587	10.223	7.128	3.095	12.494
45	9.849485	1.033561	10.803	7.639	3.164	13.388
48	.871073	.055149	11.354	8.148	3.206	14.280
51	.890503	.074579	11.874	8.657	3.217 max.	15.172
54	.907958	.092034	12.360	9.167	3.193	16.066
57	.923591	.107667	12.813	9.676	3.137	16.958
60	9.937531	1.121607	13.231	10.195	3.036	17.850
63	.949881	.133957	13.613	10.695	2.918	18.746
66	.960730	.144806	13.957	11.204	2.753	19.638
69	.970152	.154228	14.264	11.713	2.551	20.530
72	.978206	.162282	14.531	12.222	2.309	21.420
74	9.982842	1.166318	14.666	12.562	2.104	22.020
76	.986904	.170980	14.824	12 900	1.924	22.620
78	.990404	.174480	14.944	13.240	1.700	23.200
80	.993351	.177427	15.046	13.580	1.466	23 800
81	.994620	.178696	15.090	13.750	1.340	24.098
82	9.995753	1.179829	15.130	13.920	1.210	24.395
83	.996751	.180827	15.164	14.090	1.074	24.692
84	.997614	.181690	15.194	14.260	.934	24.989
85	.998344	.182400	15.220	14.430	.790	25.286
86	.998941	.183017	15.241	14.600	.641	25.583
87	9.999404	1.183280	15.257	14.770	.487	25.880
88	.999735	.183811	15.269	14.940	.329	26.177
89	.999934	.184010	15.276	15.110	.166	26.476
90	10.	1.184076	15.278	15.278	0.000	26.775

Note.—To obtain column III add to log. sin. in column II $\left\{\begin{array}{l}\text{Log. } 14 = 1.146128 \\ \text{Log. cot. } 42^\circ\ 30' = 10.037948\end{array}\right\}$ constant $\underset{1.184076.}{\text{Log.}}$ In column V are the proportionate values of half the obliquity=15.278 ft. Quantities in column VI are ordinates parallel to impost. See Record Plan, Plate XLVI.

Table 3 exhibits the method of applying the preceding principles and the use of the formulæ given above.

Explanation of Table No. 3.

Column I contains the different assumed values of the arcs whose ordinates have to be found, beginning with a group of variants of 4 degrees 30 minutes ($\frac{1}{20}$ of quadrant), followed successively by groups of 3, 2 and 1 degree variants.

Column II contains the corresponding logarithmic sines of the angles in column I.

Column III.—The numbers in this column are the logarithms of the lines *be*, Fig. 3, and are obtained by adding log. r + log. cot. Θ to each of the numbers in column II.

In this case

$$\log. r = \log. 14 = \qquad 1.146128$$
$$\log. \cot. \Theta = \log. \cot. 42^\circ\ 30' = 10.037948$$
$$\log. r \cot. \Theta = \qquad 1.184076$$

and this, added to each of the numbers in column II, produces the numbers in column III.

In column IV are the natural numbers corresponding to the logarithms in column III.

Column V.—The numbers in this column are obtained in the following easy manner: The perpendiculars *bf*, Fig. 4, divide the development *BE* proportionately to the divisions of the quadrant, and the lines *fg*, drawn from the points of intersection *f*, and parallel to *BE* will divide *BD* in the same ratio. Now, *BD* is half the obliquity of the arch, which has been before found to be 30.556 (see Table No. 1), therefore $BD =$ 15.278 and the other values of *bf* are proportional to the respective angles ε in column I, as will be evident by inspection.

Column VI consists of the differences in columns IV and V, and are the values of the ordinates *fe*.

In column VII are like proportionals of half the heading course *DF*, which constitute the abcissas *Ef*. This development, in order to be of practical utility, must be laid down of the full dimensions.

Plate XLVI is the working record of this computation.

The straight line *DF* in Fig. 3, is the development in the heading spiral, and there being n stones in the entire ring, $\frac{DF}{n}$ is width of each stone measured on the intrados. The value of n is determined by two considerations: first, it must be an uneven number if it be desired to have a central keystone; and second, it must give stones of convenient size for working. If the entire soffit were to be made of stone it would now become necessary in most cases to adjust the intradosal angle and its functions so that the spiral shall intersect both faces at the exact lines of joints. But, when bricks are used the extreme variation can never exceed half the thickness of one brick, or say, $1\frac{1}{8}$-inches, so that it is not worth while to make the adjustment in the computation, the slight divergence being unnoticable and easily made practically in laying out the spirals. It is necessary to make the width of stone equal to a definite number of courses of brick; in this case six courses were taken,

making the width of each stone 1.373 feet and the number of stones $\frac{53.55}{1.373} = 39$. The parallel lines at right angles to the heading spiral in Plate XLVI are the developed coursing spirals, and the portions of the soffit development included between any two spirals are the developments of the face edges of the stones.

Laying Out the Face Joints.

Because the intradosal ellipse, Fig. 5, coincides with the soffit face and equals it in length it follows that the lengths of the latter included between the coursing spirals are respectively the lengths of the faces measured on the ellipse, and the joint points 1, 2, 3, etc., Fig. 5, can thus be set off directly. The directions to be given to the face joints require further consideration. If the extradosal surface were to be developed in like manner other points could be similarly found in the extradosal ellipse, and intermediate points could also be obtained by intermediate developments, and it would be found that the joints would theoretically not be straight lines in the face, but curves of very light degree; it would also be found that the chords of these curves would, if produced, all intersect in a point below the center of the true circle, called the point of eccentricity; its distance depending on the angle of obliquity, being termed the theoretic eccentricity, is obtained from the following equations:

$$CO \text{ (Fig. 5.)} = \frac{\text{Cot.}^2\,\Theta}{\frac{1}{2}\,\pi}\,(r + e) \text{ or in this case}$$

$$CO = \frac{1.0913^2}{1.5708} \times 16.167 = 12.27.$$

The curvature of the joints being exceedingly light the chords practically coincide with the arcs within the limits of the stone, and it is therefore only necessary to lay off the distance CO and from the point O draw right lines through the points 1, 2, 3, etc., for the face joints.

The Angles between the Face and Coursing Joints.

The most delicate and difficult part of the mechanical execution is the exact cutting of the angles between the face and coursing joints, there being two different angles to each face stone, the angle on the right hand of one stone being the left hand angle of the adjacent stone.

Whenever the stones are cut at the bridge site there is no practical difficulty in obtaining the angles, as will be seen, and it is recommended

that this should always be done; even if it is desirable to cut the other parts of the stone, at the quarry or elsewhere, the faces should be left in the rough and cut just before setting.

In this case the angles were computed and used in making up the table of quarry dimensions (Plate XLVII) by which the stones were got out in the rough, but the exact angles were measured before final cutting by the use of a bevel gauge or shifting stock devised by Mr. McConnel, one of the engineering assistants on the work, and applied as follows:

The centers having been set, the lines of the coursing spirals were marked out upon them and also the line of the faces; in the plane of the face a tack was driven in the point of eccentricity before described. Augur holes pointing towards the point of eccentricity were bored through the lagging at the points where the coursing spirals intersected the line of face; a cord was then stretched taut, passing through and connecting the two points and projecting beyond the lagging; the shifting stock was then set to the angle formed by the cord and the coursing spiral on the lagging which was the angle required for that joint; the shifting stock was then clamped and the angle transferred to the stone.

In order to obtain the angles by computation for use at the quarry, the following method (see Table 4) was used.

Fig. 6 (Plate LII) is a graphic representation of the angle to be computed. Fig. 7, a diagram illustrating the method.

$o\,n$ is the half arch on the square, $r\,a$ being the radius, r; $r\,s = r \times$ sin. ϕ, $r\,y = r$ cos. ϕ tan. Θ, $y\,c =$ radius of osculatory circle $\frac{r}{\text{sin.}^2\,\beta}$ $=$ in this case $\frac{14}{.570^2} = 43. +$

From the points a in the semicircle of arch corresponding to joint points draw lines $a\,r$ to the center, making the angles γ, being equal divisions of the quadrantal angle. With $r\,s$ as a radius describe the arc $r\,g$, from a drop perpendicular $a\,p$; from a' draw $a'\,a''$ parallel to $r\,o$, and perpendicular $a'\,d$.

Now, $a'\,d = r\,s \times$ sin. $\lambda = y$; $r\,p = r \times$ cos. $\lambda = x$.

$$\text{tan. } a''\,y\,n = \frac{x}{y + \text{constant } r\,y.}$$

Table 4 contains the values obtained from above equations and requires no further explanation.

TABLE No. 4.

Oakley Arch. Determination of the Angles of Face and Coursing Joints.

λ Angle of Face Joint with Horizontal.		Log. Sin. λ	+ Log. 8.755, or 0.942256	Nat. nums. Y	Log. cos.	+ Log. 14, or 1.146128	Nat. nums. z	Y + 1 .01.	Nat. tan. Face Angles with vertical.	Face Angles.	
°	′									°	′
4	37	8.905736	—1.847992	.705	9.998589	1.144717	13.955	10.715	1.3024	52	29
9	14	9.205354	.147610	1.404	.994336	1.140467	13.819	11.414	1.2107	50	26
13	51	9.379089	.321345	2.095	.988489	1.134617	13.634	12.105	1.1263	48	24
18	28	9.500721	.442977	2.773	9.977041	1.123169	13.279	12.783	1.0389	46	06
23	05	.593363	.535619	3.432	.963757	1.109885	12.880	13.442	.9582	43	47
27	42	.667305	.609561	4.069	.947136	1.093264	12.389	14.079	.8799	41	21
32	19	.728027	.670283	4.681	.926911	1.073039	11.831	14.691	.8053	38	51
36	56	.778792	.721048	5.261	.902729	1.048857	11.191	15.271	.7329	36	14
41	33	9.821693	.763949	5.807	9.874121	1.020249	10.480	15.817	.6626	33	32
46	10	.858151	.800407	6 316	.840459	.986587	9.695	16 326	.5938	30	42
50	47	.889168	.831424	6.783	.800892	.947020	8.852	16.793	.5271	27	48
55	24	.915472	.857728	7.207	.754229	.900357	7.950	17.217	.4620	24	48
60	01	.937604	.879860	7.583	.698532	.844660	6.993	17.593	.3980	21	42
64	38	9.955969	.898225	7.911	9.631859	.777987	5.997	17.921	3346	18	30
69	15	.970874	.913130	8.187	.549360	.695480	4.960	18.197	.2726	15	15
73	52	.982551	.924807	8.411	.443847	.589975	3.892	18.422	.2113	11	56
78	29	.991167	.933423	8.579	.300276	.446404	2.795	18.589	.1503	8	33
83	06	.996843	.939099	8.692	.079676	.225804	1.682	18.702	.0899	5	08
87	43	9.999655	.941911	8.748	8.600332	—1.746460	.558	18.758	.0297	1	42

The actual face angles are deduced by subtracting the angles γ from 90 degrees for the acute side, and adding the same angles to 90 degrees for the obtuse side, and adding in each case the angle formed between the tangent of the osculatory circle and its chord of the length of the stone along the coursing joint, as given in table, which is simply a continuation of the process from the preceding table.

TABLE No. 4—*Continued.*

Oakley Arch. Angles of Face and Coursing Joints.

No. of Joint.	From Table No. 4.	Acute Angles.		No. of Joint	From Table No. 4.	Obtuse Angles.	
	° ° ′ °	°	′		° ′ °	°	′
1	90 — 52 29 + 3	40	31	38	52 29 + 90 + 3	145	29
2	90 — 50 26 + 3	42	34	37	50 26 + 93	143	26
3	90 — 48 24 + 3	44	36	36	48 24 + 93	141	24
4	90 — 46 06 + 3	46	54	35	46 06 + 93	139	06
5	90 — 43 47 + 3	49	13	34	43 47 + 93	136	47
6	90 — 41 21 + 3	51	39	33	41 21 + 93	133	21
7	90 — 38 51 + 3	54	09	32	38 51 + 93	131	51
8	90 — 36 14 + 3	56	46	31	36 14 + 93	129	14
9	90 — 33 32 + 3	59	28	30	33 32 + 93	126	32
10	90 — 30 42 + 3	62	18	29	30 42 + 93	123	42
11	90 — 27 48 + 3	65	12	28	27 48 + 93	120	48
12	90 — 24 48 + 3	68	12	27	24 48 + 93	117	48
13	90 — 21 42 + 3	71	18	26	21 42 + 93	114	42
14	90 — 18 30 + 3	74	30	25	18 30 + 93	111	30
15	90 — 15 15 + 3	77	45	24	15 15 + 93	108	15
16	90 — 11 56 + 3	81	04	23	11 56 + 93	104	56
17	90 — 8 33 + 3	84	27	22	8 33 + 93	101	33
18	90 — 5 08 + 3	87	52	21	5 08 + 93	98	08
19	90 — 1 42 + 3	91	18	20	1 42 + 93	94	42

See Record Plan, Plate XLVI.

We have now all necessary dimensions for the templates for the soffits and faces, for which there will be one each for one-quarter of the whole number of stones in both faces, and for laying off the face angles, which may be done either by templates or with shifting stock as preferred. It is to be borne in mind that with the exception of the variations produced by the faces all the stones on the intrados are governed by the same rules and vary only in length.

Method of Working the Stones.

Because the depth of a stone, or the breadth of its bed, invariably measures considerably more than its width on the soffit, it is the better way to begin by working the former.

The beds are portions of a spiral surface, and it may assist the imagination in considering its relations to remember that they are the only spiral surfaces in the stone; they correspond to the working surfaces of the thread of a square-cut screw. We may conceive of this spiral surface as being made up of the successive positions of a right line moving horizontally at a uniform rate along the axis of the cylinder of the arch, with one end always in contact with the axis and the other, or outer end, moving around the surface of the cylinder also at a uniform angular rate. Every position of the right line is in a vertical plane at right angles to the axis. The right line itself is vertical at the crown and horizontal in the case of a semicircular arch, at the springing lines. The rate of the angular variation to the horizontal motion is the degree of the spiral, and is measured by the intradosal angle. The ratio being constant, any portion of the surface so generated would be the counterpart of any other portion, or in other words, the spiral surfaces of all the stones are exactly alike for equal lengths.

The successive positions of the generating right line have an angular relation dependent upon their horizontal distance apart. This variation is technically termed the "twist," and it is evident that the twist for any given length would be the twist for an equal length in any other part of the spiral, no matter what its position in space might be. It is also evident that as the twist is measured by the central angles, its amount at any part of the surface is a function of the radial length, and that if we are dealing with a part of the surface, included between the intradosal and extradosal cylinders, the amount of twist is the difference

between the functions of their respective radii, or in other words, the difference between the intradosal and extradosal angles is the angle of twist δ or wind of the bed, when applied with consideration to respective lengths of the intradosal and extradosal spirals.

The mode of obtaining the winding beds, as such spiral surfaces are called, is familiar to workmen, and is done by placing two rules (Figs. 8 9 and 10, Plate LIII) or straight edges, one of which has its edges parallel and the other diverging, at a determinate distance, and then each is sunk into a draft in the stone until their upper edges are in one plane, when the under edges will be in the winding surface or bed. This done, the superfluous parts of the stone on the other parts of the bed are dressed off until a straight edge applied from one draft to the other, and in contact therewith, being always kept parallel to the soffit, shall in every part of the bed coincide with the surface thereof. It is worthy of note that this involves no more work than if the bed was a plane.

In order that the workmen may not be liable to mistake in applying these rules, and to obviate the necessity of measuring the divergence, they are connected by light iron rods with a hook at one end dropping nto a staple and with an eye at the other. These rods being of computed lengths, must always give the proper radiation and be used in correct relation, the hooks being on the parallel rule and the fixed eyes on the beveled one.

The formulas for finding the radiation between the rules, and for the divergence of the sides of the bevel rule, dependent on the foregoing principles, are as follows, wherein

l is distance (arbitrary) between rules at small end of bed.
β angle of intrados (Table I).
ϕ angle of extrados.
$\delta = \phi - \beta$.
l_2 = distance between rules at outer end $= l \times \dfrac{\text{sec. } \phi \text{ cos. } \delta}{\text{sec. } \beta}$
d = difference in width of bevel rule $= l_2 \times$ tan. δ,
In this case $l = 36$ inches, $l_2 = 37.80$ inches, $d = 2.6$ inches.

This pair of rules is applicable to every stone in the arch, and would be as well if the entire arch was of stone.

One bed having been worked, the soffit must next be obtained from it, and to effect this the "saddle-mould," Figs. 12 and 13, is used. As this device, although so simple, accurate and reliable, is not generally known, instructions for making it are here given. Prepare two wooden

moulds, as shown in Fig. 11, where AC is the radius of the cylinder, and DB is its thickness or depth of voussoirs.

The stock of the mould must be made to fit the arc of the circle of the soffit and may be of any convenient length, preferably as long as the stone is to be wide, or a little longer. BD is made equal to the depth of voussoirs, with both edges radiated to the center. These two moulds should then be turned back to front and framed together as represented in the perspective Fig. 12 and in bottom plan in Fig. 13, being so placed that the angle ACB shall equal the complement of the angle of the intrados. This being done, the edges of the two blades BD and CE will exactly coincide with the spiral bed of the stone already worked by the twisting rules, and the stone being placed with its soffit side uppermost, let the saddle-mould be inverted and applied with its blades in contact with the worked bed and the curved strip BC, lying along the soffit of the bed, as shown in Fig. 13. Then let a line be drawn on the stone along the stock AC, and this line, so drawn, will be at right angles to the axis of the cylinder. Let another line be drawn along the side AB parallel to the axis of the cylinder. Remove the saddle and let chisel drafts be sunk in the soffit on the line CA, to fit the curve of the stock, and also on AB (which will be perfectly straight), to fit the side AB, so that when sunk to the proper depth, the saddle on being re-applied with its blades to the previously worked bed, and the diagonal curved strip to the arris of the soffit, the stock CA and the side AB shall all be exactly in contact in every part at the same time.

Two segmental pieces, each as long as CA and of same radius, are used, one to apply to draft AC and the other upon a line GH, Fig. 13, drawn at a convenient distance and parallel with their centers on the line IK. The second segment is then sunk in a chisel draft until its upper edge is out of wind with the first segment. This being done, the superfluous stone on the soffit is dressed off until a straight-edge applied parallel to AB coincides with the soffit in all parts.

The other arris DF of the soffit should then be gauged and knocked off parallel to LM, and then the saddle turned about, and by reversing the former procedure the other bed is cut. The ends of all voussoirs except those forming the face are worked square on the soffit, and the other parts of the ends of the stones, technically called the heading joints, must be worked to fit the radiation of the blades of the saddle-mould where the entire arch is of stone.

The saddle-mould is applicable, like the twist-rules, to every stone in the arch, and one mould will be sufficient for the use of a number of stonecutters. Plate XLIII is an enlarged view of the mould in use, and the segment blades and twist-rules may also be seen.

After the beds and soffit are cut it is only necessary to apply the proper face angles to each bed after having marked off the soffit face by means of its template. The remainder of the stone-cutting for the rings contains no unusual features. Plate XLIII shows a specimen stone.

The Impost and Saw Teeth Skew Backs.

Where stone is used throughout the soffit the impost is so cut as to project into the curve of the arch sufficiently to intercept one width of voussoir. When brick is used two ways are open : either to project the impost as before to intercept as many courses as the ring stones head, or to cut individual checks in the impost, one for each course of brick.

Where the obliquity is not too great and the necessary overhang not beyond the limits of conveniently-sized stone, the former plan recommends itself.

In this case, however, it was deemed better to use individual checks. Plate XLIX shows their arrangement. See also Plate XXXVIII.

They are easily designed by use of the intradosal angle.

Locking the Lower Ring Stones.

It is evident from inspection that the beds of the lowermost ring-stones on the left-hand side of this arch (see Plate XLI) are so inclined to the horizon that they would have a tendency to slide out, and that the elevation at which this tendency disappears is quite considerable. Without going into the analytical discussion, which is too lengthy for this paper, it may be stated that the point above which theoretic stability exists is, in this case, at a distance above the springing line = to r sin. 20°, approximately. This practically involves the first five ring stones, and is met by two devices. The first consists of eye-bars inserted between the stones, to which they are secured by iron dowels, and extending back 12 feet into the brick-work, where they are anchored. The object of these is chiefly to temporarily secure the stones during construction and until the masonry has become coherent. The principal reliance is upon an original device by which the wing

walls are made to act as buttresses by means of locking-stones which extend into the wing walls and are interposed in front of the ring stones. The ends of the imposts are carried out square to obtain additional strength and size for the locks. This arrangement is plainly seen in the face views of the arch, Plates XLI and XLV, and is shown also in the ground plan Plates XLV and XLIX, and in the diagrams of stones 1 and 2, Plate XLVII. The similar construction on the obtuse side is resorted to for harmony, but is not there required. The locks terminate well above the danger point. By their use the face stones cannot slide without displacing the wing walls.

The first stone on the left of the face marked No. 1 on the diagram sheet required some special cutting, but as it is simply a combination of face and impost in one block and involves no special principles of stereotomy it need not be further described.

Laying Out the Coursing Spirals.

Plate XLVIII is a diagram showing one complete spiral of the intrados with ordinates of points of its true position, referred to the vertical plane passing through three points, which are its intersections with the two springing lines and the element of the cylinder at the crown. The horizontal projection of the spiral being a curve of sines, the computation is a simple one. The intermediate points to which they apply can be set off on the lagging of the centers by means of a transit instrument set in the vertical plane and plumbing or squaring down to the cylindrical surface. Any spiral can be set in this way, it being only necessary to fix its intersection on the crown.

The heading spirals may be laid off in a similar manner if desired, though scarcely necessary in the case of brick work.

Plate XLIX shows the horizontal projections of both coursing and heading spirals. By applying a flexible, narrow straight-edge to any two points in the spiral on the lagging, and bringing the straight-edge into continuous contact with the lagging, the full curve can be traced on the lagging with the utmost nicety and exactitude, and this should be done at frequent intervals so that the brick joints can be made to conform throughout to the theoretical curves, Plate L. To secure an accurate cylindrical surface the planks for the lagging should be run through a forming machine to give them the proper convexity.

NOTE BY THE AUTHOR.

It is interesting in this connection to consider the relative cost of the skew construction. Before adopting this form comparisons were made with others which might be used.

One plan considered was to make a square ended arch with face ramps extending up on the right hand sides to meet the slope line of embankment. This would have involved a lengthening of the left-hand impost and a material increase in height and dimensions of the wing walls, besides the additional masonry on the ramps. Another plan was to lengthen the barrel and use a skew sufficiently slight to justify square joints. A third was to use offsetted ribs.

The result of the comparison showed a material economy in expense in favor of the adopted plan. The allowances made to the contractor for extra cutting on the imposts and wing stones, for a superior quality of stone, for extra labor on centers and for other incidentals, made a total increase of cost in the masonry item of about 50 cents per cubic yard, making it cost a very little more than $10.50, $10.00 being the price paid on the same section for first-class straight work.

In either of the make-shifts proposed the cost of the additional quantity of masonry and piling foundation necessary would have largely exceeded the extra price paid, which was somewhat greater than it otherwise should have been on account of the difficulty of working in such close quarters. In ordinary cases of average dimensions, with no restrictions such as there were here, it is a reasonable expectation that the average cost per yard of the entire masonry in a skew construction will be increased by from three to four per cent. by the heliocoidal treatment.

Any make-shift adopted to avoid departure from straight joints, even the very primitive one of reducing unit pressures by thickening the arch, will generally cause an increase in quantities largely in excess of this percentage; so that there is true economy as well as theoretic security in the scientific method. There are many cases of slight skew where it is not worth while to deviate from straight joints simply because they are well within the permissible angular limit. As an example, the author designed and erected the Montgomery Arch, also on the Cincinnati and Richmond, of same dimensions as the Oakley Arch, but whose angle of obliquity was 68° (skew 22°) where straight joints were adopted.

There is no record of the early oblique arches. The period when they were first introduced is not known. It was while mechanical masonry was a craft and the master mason was an engineer, who worked but did not write. His methods were transmitted by example and his tools and devices were the secrets of his art.

The art appears to have been known early in the sixteenth century. Buck quotes a reference to Vasari, who mentions a bridge of this kind erected near Florence in 1530.

But when in the present century Chevalier Mosca, Engineer and Architect to the King of Sardinia, contemplated a skew arch over the Dora Riparia, near Turin, it is recorded that "he felt too that the art, although not of recent origin in Italy, does not afford at this day proper means of executing such a work satisfactorily on a very large scale."

Since that time the modern engineer has been evolved and a number of beautiful examples of skew-arches are to be met with. In our own country, however, they are comparatively rare. Under the New York approach to the Brooklyn Bridge is a most excellent instance of the segmental arch in brick with stone faces; while at Reading, Pa., on the line of the Lebanon Valley Railroad, is a well constructed skew in stone throughout. But there are so many cases where fine opportunities to apply correct principles have been neglected or ignored that it would appear as if American engineers were too much engaged with the weightier matters of their pursuit to find time to adopt them. The extra labor they entail, whatever it may amount to, comes upon the engineer only, and he should not shirk it because a crude method is easier and requires less time.

There are men eminent in the profession who condemn the refinements of design, and who regard the use of spiral joints as sophomoric, forgetting that their object is not fantastic, but a simple arrangement of individual stones or bricks, so that the stresses that come upon them may be as nearly as possible at right angles to their resisting faces.

It is quite true that the spiral joint is not necessary for the part of an arch, considered by itself, in which the abutments face one another directly—in which lines at right angles to the axis meet both abutments, but there may be and are instances where there is no such part—where the distance between the faces is less than the obliquity. In such cases, and in the projecting portions of all skew arches, the reactions of the abutments must travel towards each other either in the shortest direc-

tion inclined to the cylinder or by a succession of steps producing shearing strains. If the faces of the stones or bricks are square to the path of the force we have the best conditions for stability; otherwise we must depend upon the transverse strength of the material and largely on the adhesive strength of the mortar, which may at any time be impaired by one of several natural causes.

We are sometimes told by the opponents of engineering delicacy that "brick-work becomes a monolith;" it may, in time, and often undoubtedly does, but it does not always remain so, and there is no reasonable assurance in any case. Brick walls have a better chance to reach the monolithic state than have arches, but, nevertheless, it is not considered good practice to lay the courses in the wall inclined to the force of gravity, in fact, on the contrary, great pains are taken to keep them level.

The heliocoidal treatment is not a perfect one, for it fails to provide joint surfaces normal to the action of gravity near the springing line, but it does afford security to the vulnerable parts of the arch, and its uniformity and preciseness of application commend it as against the complicated variations in the theoretically perfect equilibrated arch. In structures for the passage of water courses, where the soffit is hidden from view, and where the marginal loads are relatively light, as under the toes of embankments for instance, the engineer may well hesitate before encountering the problems of skew construction, and may often avoid them with sound discretion, but in a case where the opening is to be used for public travel, beneath heavy moving loads, where symmetry is desirable as well as strength and unsightly features should be excluded, the heliocoidal treatment approximates more nearly to all the requirements than any other, and by its adoption it is possible to secure the full measure of the strength of the material used, which is another name for economy.

DISCUSSION.

DESMOND FITZGERALD, M. Am., Soc. C. E.—Would it not have been a little better, as a matter of artistic design, perhaps, to have made the wing walls rather more at right angles with the face of the arch? Was that considered?

Mr. CROWELL.—I am glad Mr. FitzGerald has asked that question, because it had occurred to me, after the paper was written, that it would be well to refer to it.

It would have been desirable to do so if there had been room on both sides, but on the north side the creek interfered. The fact that the difficulties of the foundation increased towards the creek made it desirable to keep away. On the south side of the arch the creek did not thus interfere, but it was not thought desirable to make a variation. The wall is designed with reference to the angle; there is no loss there.

D. J. WHITTEMORE, Past President Am. Soc. C. E.—Has Mr. Crowell made any estimate of the cost of this structure as compared with the cost of an iron girder?

Mr. CROWELL.—The fact that the railroad embankment was to be raised 11 feet higher, at an undetermined date, perhaps remote, perhaps near, made it imperative that we should adopt a covered structure that would not be subject to future alteration. The fact is, however, that I did make such a comparative estimate, and there was less difference than one would suppose. The dimensions of the foundation and sidewalls would have been increased, because of the additional height required.

Mr. WHITTEMORE.—There is one feature in relation to stone arch bridges, built in our Northern climate, to which I wish attention called, as it refers to a matter that may be unknown and not generally guarded against by members who have practiced in a mild climate. I have found when arches are sheeted with brick, and a joining made by toothing same with ring stones on which the parapets are built, that after a few years, through the lateral expansion and contraction of road-bed by reason of great changes of temperature, the ring stones become separated from the brick sheeting from 1 to 2 inches, therefore I advise that wherever brick sheeting is used, the same be extended to the very end, leaving off ring stones entirely. This may not be esteemed artistic, but it is certainly better.

HERBERT M. WILSON, M. Am. Soc. C. E.—In a recent trip to India I saw a skew arch of peculiar construction. It was not a simple road-bridge arch, but a combination of a masonry road-bridge and a canal regulator, at the head of the great Ganges canal. The skew form was adopted

because the canal was headed at such an angle with the river that a regulating bridge at right angles to the line of the canal would have caused a great deal of silting at the mouth of the canal, and it became necessary to make the upstream face of the line of regulating gates parallel to the course of the river. At the same time it was necessary to have the bridge at right angles to the course of the canal on the lower side. The British engineers in charge made computations similar to those we have just heard described, but they felt so uncertain as to the result that they constructed a very large model previous to undertaking the final work. The model was constructed of brick, 40 feet high, perhaps, and possibly as long, showing only a portion of the structure, including one regulating gateway. The foundation would have to be in gravel extending for some depth below the bed of the river. A peculiarity is that this skew arch is to be double-decked, the lower deck being intended to carry the roadway and the upper one being required to give head-room for operating the regulating gates. Another peculiarity of this skew arch is that it is to be constructed across the Ganges canal, where it is 140 feet wide on bottom and 220 feet wide on top, thus requiring a structure triangular shaped in plan, being only 20 feet wide at the east end, and about 150 feet wide at the west abutment. The mass of masonry which will be required in its construction is thus enormous. When seen the stones were all cut and shaped, partly from designs, partly from the model, and the structure was just being commenced. The figures which I have just given of the height and other dimensions of the skew arch are as I recollect them from guesses made by looking at the model.

G. Bouscaren, M. Am. Soc. C. E.—Why is the angle of 40 degrees 30 inches the most unfavorable angle?

Mr. Crowell.—In reference to the point of eccentricity I said that its position had a bearing on the question of strength, and it is on that account that an angle somewhere between 45 and 42 degrees produces the combination of elements which is the most insecure.

It is found that at the angle of about 42 degrees 30 inches the greatest inclination of the beds of the stones is produced; before you get to this point you have not quite reached the angle of repose; when you get beyond it the increased obliquity and the lowering of the point of eccentricity begin to compensate. It happened that the angle of greatest insecurity is the one we had to adopt, because the alignment had been previously established.

Mr. Bouscaren.—Mr. Crowell stated that the additional cost, due to the special character of the work, was fifty cents a yard. What quality of stone was used?

MODEL OF SKEW ARCH FOR REGULATOR AT HEAD OF GANGES CANAL.

Mr. Crowell.—An average cost for the entire masonry, including the foundation, face, backing and extra work, was $10.50 a yard. The expectation was that limestone which came from the most accessible quarries would be used. Most of the structures on this piece of railroad were built of this stone. The contract price for ordinary first-class work was $10; on an adjacent section we paid $11. When it became necessary to adopt a stone for the ring-stones where the angles were very acute, we decided to use oolitic limestone, which we got from Indiana. We had to get it in larger blocks and had to pay an extra price for it, that added quite an increment to the cost. We had to make other allowances to the contractors because of the difficulty of working with the trains passing continually overhead and the difficulty of getting close to them. It is a perfectly fair statement that in ordinary circumstances, without the difficulties of this special location, the cost per yard would not increase the cost of the entire structure more than 3 or 4 per cent.

Mr. Desmond FitzGerald.—Did the contractor pay for the transportation of the stone?

Mr. Crowell.—He paid all the cost. In that section of the country we had quite a large amount of masonry, and the prices that we paid ranged from $8.50 to $11.50 a cubic yard. The custom was to average the price on all the work.

J. Emil Hilgard, M. Am. Soc. C. E.—Could you tell us what the cost of cut stone would be instead of brick work? I understand that the prices given were the average paid for cut stone and brick work. What would be the value of the brick work and of the cut stone?

Mr. Crowell.—I could not give figures that would be of value without the books. I would be drawing largely on my memory.

Mr. Hilgard.—How much camber did you give to the centers?

Mr. Crowell.—None, and we had no settlement.

Latham Anderson, M. Am. Soc. C. E.—I have recently used some of the stone from the Salem quarries. I would like to know the strength of that stone and of ordinary Indiana flat rock. I have found that stones of that kind spawl pretty badly, even with not very great strains upon them. The particular stone I speak of was from the Salem quarry.

Mr. Crowell.—There were no comprehensive tests made of the relative crushing strength of the stone, but it was evident that layer stone that came from the limestone quarry would probably be strong when lying upon its natural bed, and might not be strong when lying in the other direction. It was necessary to have a stone that was strong in both directions. The oolitic stone proved such a stone. It was necessary to get a stone of large dimensions in two directions. It was desirable to

contain in one mass the first stone in the ring and the first stone of the impact.

O. H. Landreth, M. Am. Soc. C. E.—About a year ago I made some tests of this Salem stone. In about 15 cubes I got a strength of about 5 000 pounds to the square inch. Two of these cubes were, by mistake, put in the machine vertically, and the results were entirely different. In the horizontal case the fractures were ideal almost. I never saw any more nearly ideal cases than the 13 cubes crushed in that way. In the other case the fracture was entirely different. It ran at an angle of about 30 degrees with the bed of the machine, and crushing occurred at a much earlier stage, showing that the stone was not homogeneous in the two directions.

F. Collingwood, M. Am. Soc. C. E.—Mention has been made in the paper of skew arches in the East River Bridge approach in New York. It so happened that these came under my own charge, and I may say that in no instance in that work was there any dodging of the problems presented. Specimens will be found of groined arches, coned arches and skew arches.

At North William street, the arch has a vertical length between face walls of 100 feet, a span of 40 feet, and rise of but 5 feet. The greatest difficulty was not in the amount of skew, but to prevent slipping at the springing planes. As the weight above these planes was small, a horizontal depth of 30 feet was given to the abutments and counter arches turned underneath to lighten them below.

The small arches in the walls facing Pearl and Cherry streets were skew on the outer face and square on the interior. The skew face stones were carefully cut to correct lines and set, and then the lines for the brickwork completing the inner end of the arch were sketched in on the centering in such a manner as to bring the joints at right angles to the face.

One difficulty, not mentioned by Mr. Crowell, in setting the face stones in an arch of high skew is the danger of spawling from the acute angles, as the pressure is developed. To avoid this it is necessary not to fill the joints at the face, until the cement has had a chance to set, after which the joints can be cleaned out and pointed. It may be said that this precaution should be observed in laying all masonry which will be subjected to a considerable increase of pressure after setting.

Mr. Crowell.—In reference to pointing I consider it very important not to point at first. In a brick arch (60 feet span) the inner layer of bricks was crushed all along one abutment; the inner layer was carefully pointed, while in the other six rings of brick the joints were more open. When the center was struck the inner course at the skew backs was crushed and had to be replaced.

James B. Francis, Past President Am. Soc. C. E.—There seem to be two causes for the spawling; one is that the beds are cut more perfectly near the face of the stone than in other parts, permitting a more perfect contact at the arris and consequently a greater strain than at other parts of the bed. The other cause is that the stone offers less resistance to fracture near the arris. The old Greeks understood this, and in their most perfect works, in which little or no mortar was used, it has been found that the beds, near the arrises, are just enough slack to prevent contact. Modern masons understand that they can prevent spawling by a mortar joint.

Charles Paine, Past President Am. Soc. C. E.—It might be interesting if the list of references in this paper were extended to cover other references made by members to arches with which they are acquainted. There is a very handsome stone skew arch of 50 feet span at Silver Creek, upon the Buffalo end of the Lake Shore road, which was erected under the direction of Mr. Enright, and built by Mr. Ensign, a Fellow of this Society.

A Member.—I can add another which has come to my notice, a skew arch crossing the tracks of the St. Paul Railroad. It is built of red sandstone. It has been described in *Engineering*.

A. Fteley, Vice President Am. Soc. C. E.—In coming here yesterday, we saw that on the Pennsylvania Railroad they are constructing several bridges in skew form, and I noticed that their construction was different from what has been shown to-day, the oblique lines of the bridge abutments being broken in echelons, which support a series of ribbed arches independent of one another and parallel with the axis of the track. Is there any one present who can tell why this construction has been resorted to?

Although the preparatory work of building a skew arch is costly, it seems to me that, on the whole, the total cost would not largely exceed that of a straight arch if the preparations were properly made in advance for the cutting of the stone.

I happened to come along this same road a few days after the Johnstown catastrophe, and was appalled at the amount of destruction on the streams. Some twenty-five or twenty-six bridges were carried away on the Juniata. Such disasters illustrate very forcibly the necessity of avoiding obstructions in waterways.

During the course of some experiments that I was making on the flow of water through different channels, I found that in a tunnel built for conveying water, in which the floor was made of smooth concrete and the sides were left ragged as the rock excavation left them, the flow was very much retarded by the roughness of the walls. When compared with a smooth brick channel, it was found that the percentage of loss in flowing capacity was as much as 40 per cent.

A MEMBER.—In what dimension of channel?

Mr. FTELEY.—Ten feet wide. There is no question that the slightest irregularities in the sides of a channel will cause eddies and will disturb the flow. This is so true that a mere wash of Portland cement applied to a brick surface with a brush, will give from 2 to 3 per cent. additional flowing capacity over the smooth brick work.

Considering the diminution in flowing capacity due to the irregularities produced by the ribbed arches, I do not see that in point of cost they present any advantage. It may be added that on one side of the bridge the projections of the ribs might easily stop trees or similar obstructions carried through the bridge in time of flood and produce a dangerous accumulation of water in front of the waterway.

Mr. CROWELL.—Mr. Fteley has called attention to one reason why this form should not be adapted for a water course. Another reason why it should not be used for any purpose is that the diameter here (referring to diagram) is reduced. In the third place, the use of these parallel ribs renders it impossible that there should be a bond between these ribs except in parts. All the different courses of brick that compose each ring can be bonded together only at one point in the middle of the arch. Each ring can be bonded to the next ring in one place and that place is changing all the time.

Mr. FRANCIS—When the skew is not too great, say not exceeding 30 degrees, I have made the beds of the voussoirs parallel to the axis of the arch, the faces being cut to conform to the spandrels.

GEORGE E. THACKRAY, M. Am. Soc. C. E.—On their visit to Johnstown the members can see a heavy stone arch bridge, built with separate rings as shown by Mr. Fteley. Two of the rings of this bridge at Johnstown were badly injured by the flood and fire without injuring the others. The two injured rings have lately been replaced.

P. F. BRENDLINGER, M. Am. Soc. C. E.—I have erected a great many skew arches, but never any on the principle or method so well and elaborately described by Mr. Crowell. I was highly entertained by Mr. Crowell's paper, and I have no doubt that all the members present, as well as those who are absent but who will read the paper, feel as I do, that Mr. Crowell deserves an immense amount of credit for the able and elaborate manner in which he has handled the subject.

All the skew arches that I have constructed are what are termed "false skew arches." The barrel of the arch being built as if it were a right-angled or square arch, the only difference between the false skew arch and square arch being at the ends, where the faces of the ring stones in the false skew arch are cut to an angle, while in the square arch they are cut at right angles to the beds, the facade being at right angles to the axis of the arch.

I have been engaged in constructing these skew arches since 1877. The first one I constructed was in 1877 on the Pittsburgh and Lake Erie Railroad at Beaver Falls, Pa. It was designed to carry a double track of the Pittsburgh, Fort Wayne and Chicago Railroad over it, the Pittsburgh and Lake Erie Railroad passing through it. The skew or obliquity with the Fort Wayne Railroad was about 60 degrees. The entire arch was built of stone. When the springing line of the arch was reached, the centers, to carry the arch during construction, were erected to the extreme points of the arch, so that at each end of the arch there were, so to speak, some parts of the centers not built upon. I can describe this perhaps somewhat clearer by a sketch of another arch which I will describe further along. In the sketch the lines, *A B* and *D F*, represent the end faces of the arch. The centers used in the erection of the arch extended from *B C* to *D H*. The lagging was then placed on the centers, except at the extreme ends beyond the arch, where only sufficient lagging was put on to brace the centers. I well remember the conversation I had with the chief engineer, Mr. F. H. Saylor, in reference to the plans I was to prepare, of the ring or face stones for the contractor. He dwelt at least an hour on the details of development. I listened carefully and conceded the correctness of his ideas, but I knew that no matter how carefully and accurately my plans were executed, the contractor building the arch would surely make a mistake, or probably a number of them, as my experience with him was that the plans of every piece of masonry he built for me he literally smoked in his pipe. So all I gave him was one template (tin) by which he cut all the sheeting or arch stones.

By means of a transit I established the lines, *A B* and *D F*, on the lagging, with tacks about 6 inches apart, then drew pencil lines from tack to tack. The sheeting stones were then laid in courses, and the length of ring-stone required for each course was then measured on the centers; the stone was then cut on the ground, the proper obliquity being determined by measuring the lengths of the top and bottom bed on the centers, and applying these lengths to a stone on the ground and cutting it to the measurements. The result of this method insured perfectly true and accurate work; in fact the elliptical curve showed up against the sky as a background as perfect a knife-edge as can be drawn. There were no irregularities or misfits.

This arch stood twelve years without showing any signs of weakness, and, no doubt, it would have stood for ages, but owing to the usual lack of penetration in the near future, it had to be torn out, and is now being rebuilt for a double track. On the Pennsylvania Schuylkill Valley Railroad, from Port Clinton to New Boston, a distance of 25 miles, I designed and constructed sixteen arches from 16 to 60 feet spans, ten of them being built on the false skew plan, the obliquity varying from 40 to 80 degrees.

I will describe the one with the smallest angle or maximum skew: This is what is known as the "Shoe Factory Arch." It is located at the northern end of the borough limits of Pottsville, Pa. Fig. 1 shows a plan of the arch with the positions of the tracks and edge of road-bed or top of slope; also the bottom of slope or embankment. It will be noticed that the span of this arch at the spring line is 20 feet, while the skew or obliquity is so great that the overhang or projection of the arch from a square is 24 feet.

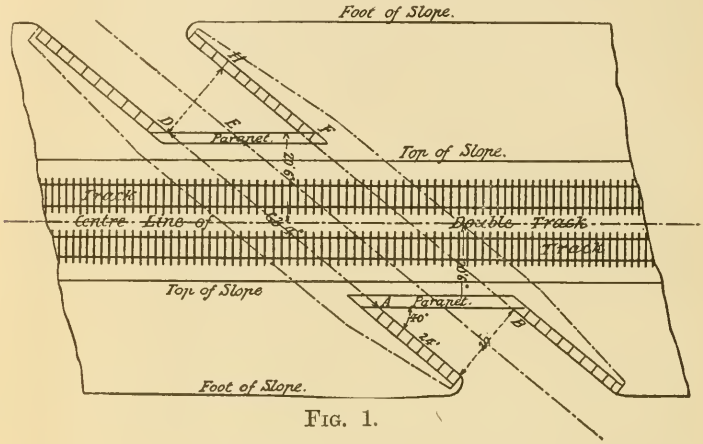


FIG. 1.

If I had read Mr. Crowell's paper before building this arch I probably would have been afraid to make the attempt, for according to what I have listened to, this arch has no business to stand up; but it is there to-day, has been under severe strain and test for the last five years, and, no doubt, it will continue to perform its functions for a great many years. The barrel of the arch is built with brick and the ends with one ring of Hummelstown brown sandstone. The use of this stone for the ring stones was a mistake, as the stones in various parts of the abutments of the arch are cracked. The worst cracks are found in several step-stones in the wings where there is no weight whatever on them, showing most conclusively that there were some inherent defects in the stone. So far there is but one stone in the ring stones showing a slight defect or spawling off, which I noticed before any weight was put on the arch, and which may have the same cause as the cracks in the step-stones. Fig. 3 shows the method I insisted on in arranging the ring stones, *i. e.*, each stone must be long or deep enough to overlap the intersection of the extreme bed of its neighbor, with the face or plane of the arch, about 10 or 12 inches, thus preventing the points of the stones from breaking or spawling off, and transferring the strain, step by step,

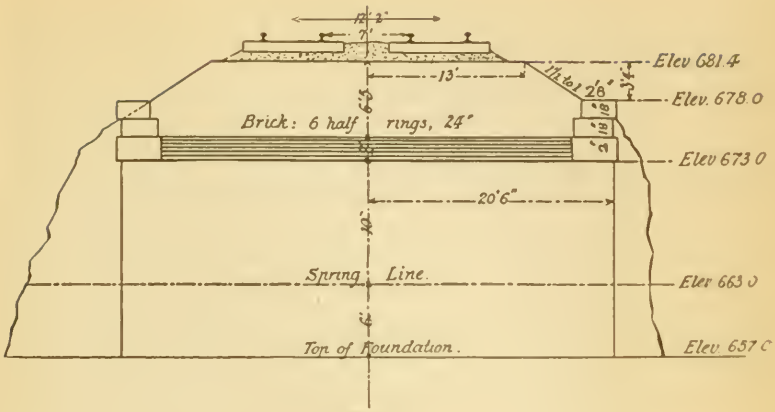


FIG. 2.

as it were, to the abutments. Fig. 2 shows a cross-section of the arch at right angles to the center line of the railroad. By a casual glance at Figs. 2 and 1 it will be noticed that there can be but little pressure on the end of the arch or ring stones, as the combined weight of the embankment, tracks and trains is too far away from the face of the arch to have much effect on the bracket-like end. The ends of the arch have little more to carry than the weight of the stones and the small part of the embankment in the slope. The whole arch acts as a monolith, and will transfer the strains to the abutments whether the stones are laid with the axis of the arch or parallel to the center line of the railroad. The only drawback the false skew arch has is, if settlements or cracks

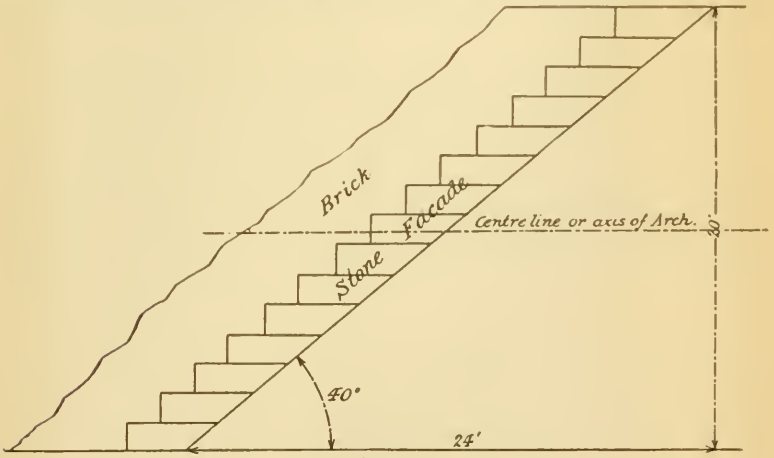


FIG. 3.

should occur, caused probably by a bad foundation, the wiseacres, as usual, would say, "I told you so," blaming it on the principle of construction. I have carefully examined the ten false skew arches above mentioned, and have found no indications of any weakness. One great mistake in the construction of arches is in making the barrel too short, thereby rendering it necessary to have a high parapet wall, which acts as a retaining wall, with the barrel of the arch as a foundation. I know several such cases on another railroad within a few miles of the "Shoe Factory Arch," where the arches were built at right angles to the railroad, having high parapet walls, with the result that the arch was literally separated at both ends, great yawning gaps or chasms showing at the corner of the arch. Had these arches been false skew arches the principle of construction would have had to carry the blame for these cracks.

The great advantage of the false skew arch over ribbed and heliocoidal arches is the great saving in the first cost, simplicity of construction, and accuracy in erection. I am confident that if plans of a false skew and of a heliocoidal arch are presented to the general run of contractors, the former will be built from $1 to $2 per cubic yard cheaper than the latter.

All things being equal, the simplest method for the engineers in designing and the contractors in constructing is always the best. The Oakley arch is a very beautiful piece of construction, and no doubt exists in my mind as to its stability, but unless the company which engages me to build arches desires to be lavish in its expenditures, I shall certainly not build them in this way.

Mr. Crowell.—I cannot pass the opportunity by for saying that one prime object in writing this paper was to demonstrate that the form which Mr. Brendlinger has just advocated is not the simplest. The heliocoidal method is the simplest. The reliance I have for making that assertion is that all the stones are exactly alike. They are not alike in their face, but the variations are in the face angles and in the size, and the one pair of parallel rules applies to every stone in the arch. It is not necessary to have a skilled workman. At Cincinnati we had to hire in the ordinary market, and it was not a very good market, and it was not a very remarkably skillful contractor, but we took the ordinary workmen just in that way, and I think there were two miscut stones out of 39 ringstones; all the workmen were working with the same saddle-mould.

As for the extra labor required of the engineer, I can say that the four tables that constitute the data in this paper were all there was. I was very busy at the time and had a good many miles of work under construction, and was laid up in my house with a broken arm; I made these tables in the evenings, and they were then given to an ordinary draughtsman who could not have managed intricate drawings.

The same simple method will enable any one to adopt this construction, if necessary. I do not say that in all cases this method should be adopted. On the same road we had another arch of the same dimensions, except that the skew was not so great, where the method Mr. Brendlinger advocates was employed.

I want to ask Mr. Brendlinger one question. I some time ago received a very beautiful bound volume of blue prints that contained the record of Mr. Brendlinger's work on the Pennsylvania Road. The volume contained the representations of several skew arches, and I looked them over with great care and admired the way in which Mr. Brendlinger had done the work. But in one which I noticed the angle of obliquity is 35 degrees, but Mr. Brendlinger had put the arch at an angle of 45 degrees; he added 9 feet at each end to the length of the structure. I want to ask Mr. Bredlinger if he made that angle because he did not care to run the risk of going to 35 degrees ?

Mr. Brendlinger.—The angle of 35 degrees for the arch was changed to 45 degrees for the reason that I concluded it was too bold; and besides that, it was a very flat arch, bringing considerable strain or pressure on the ring-stones, rendering them more liable to crack. I would not go below 40 degrees; it makes too sharp an angle. The ring-stones will spawl off if you go below that.

Mr. Crowell.—You say you made the limit 45 degrees.

Mr. Brendlinger.—No; 40 degrees.

Mr. Crowell.—You built the arch 45 degrees; you wanted 35 degrees, but concluded that was too steep; but I think you established a practical limit in that case of 45 degrees. I think you did well; but if you wanted to you could have built the arch at 35 degrees by using the heliocoidal method.

Mr. Brendlinger.—Mr. Crowell mentions an increased cost of 50 cents a cubic yard. How many cubic yards are in the abutments ?

Mr. Crowell.—The entire corner was 3 593 yards. The contractor had to go out of his way to get the limestone from a distance, and allowances were made for extra work on the centering, which really amounted to quite a great deal, because of the interference of trains with the placing of the centering. The allowance was more than in ordinary circumstances would be due to extra work; it amounted to about 50 cents on $10—about 5 per cent.

Mr. Brendlinger.—The larger the abutments are the less will be the cost per cubic yard. If a railroad company wishes to build arches (on this principle) with very small abutments, the arch stones being difficult to get, I feel confident that the majority of contractors would certainly examine those arch stones very carefully and make a very high bid.

Mr. Crowell.—The wing walls are not carried up to the top of the parapet and the abutments are of minimum dimensions; I therefore think that this is a moderately fair representation of the average. I agree that it depends on the quantities of the entire arch. As I said before, if there was no wing wall and if the arch was very short, this proportion of extra cost because of skew would be very much larger. I tried to give an average. In this case we had to pile the foundation—a difficult foundation—and every additional size given to the dimension of the work increased the cost very greatly. It was demonstrated, in this case, that there was economy in doing as we did. I am not at all an advocate of always using a skew construction, but I am in favor—and I think it is a correct principle—that when you *have* to build a skew construction you should place the material so as to get the best results from it. The nearest you can get to the normal the stronger and better.

M. J. Becker, Past President Am. Soc. C. E.—The Oakley Arch, the construction of which is so ably described in Mr. Crowell's very interesting paper, is located upon a new line of railway, of which I had general charge as Chief Engineer. The plan upon which the arch is built was very fully considered in comparison with other designs, and was finally selected, I will say, rather reluctantly, and only upon the assurance of Mr. Crowell, that he and his assistants would be willing to undertake the elaboration of the details, and give the work that careful supervision which would, undoubtedly, be necessary in the execution of a structure with whi h the ordinary stonecutters and masons would likely be found wholly unfamiliar.

From a purely engineering standpoint the work is an entire success, and Mr. Crowell and his assistant, Mr. M. P. Paret, M. Am. Soc. C. E. (who published a very full and exceedingly interesting account of the details of the construction of this arch in the "Journal of the Engineering Society of the Lehigh University," in February number, 1888), deserve great credit for their share of the work.

From an economical stand-point, however, the design itself, as well as Mr. Crowell's conclusions, expressed in the note on page 174, appear to me fairly open to adverse criticism. I am now satisfied that the method adopted for supporting the track and maintaining the traffic of the Cincinnati, Washington and Baltimore Railroad during the construction of the arch, by means of alternately shifting the temporary trestle supports proved in the end much more expensive than the introduction of a substantial truss or plate girder bridge across the gap would have been, which would have given at once ample and secure working space beneath, and would have enabled the Cincinnati, Washington and Baltimore Railroad to pass over the opening without helping engines and without a material reduction of speed. The truss or plate girder bridge could have been used elsewhere after serving this temporary purpose. There would have been no difficulty in driving the foundation

piles for the arch on the sides of and beneath the temporary bridge, the lower chords of which would have been 42 feet above the tops of the piles when driven, while the piles themselves were only 21 feet long.

The actual cost of building, shifting and maintaining the trestles, together with the amounts paid to the Cincinnati, Washington and Baltimore Railroad Company for helping engines, crews, watchmen and track labor amounted to $9 732 89.

The total cost of the entire arch is as follows:

3 593 cubic yards first-class masonry at $10	$35 930 00
13 034 feet lineal piling in foundations at 38 cents	4 952 92
23 621 feet B. M. timber in foundations at $22.	519 66
2 254 pounds of iron at 2½ cents	56 35
4 283 cubic yards earth excavation at 23 cents.	985 09
6 915 cubic yards loose rock at 38 cents	2 627 70
3 398 cubic yards wet excavation at 90 cents	3 058 20
135 780 feet B. M. timber in trestles at $19	2 579 82
1 206 days' labor on trestles at $2 50	3 015 00
Engine hire, helping trains	4 138 07
Labor, maintaining Cincinnati, Washington and Baltimore track	1 143 30
Extra cost, stone-cutting	1 833 00
Total	$60 839 11

This large aggregate expense is chiefly due to the troublesome character of the foundations, to the cost of the temporary supports and to the exactions of the Cincinnati, Washington and Baltimore Railroad Company, who not only insisted upon the immediate provision for a double track road-bed, but also for an additional lengthening of the arch to enable them to raise their tracks 11 feet above their present elevation.

While the price of $10 per cubic yard for the masonry is not high, still it is higher than ordinary rectangular bridge masonry would have cost; and while it is true that even after adding the extra allowance of $1 833 to the cost of the masonry, the ultimate price per cubic yard only foots up $10 50, I do not think that Mr. Crowell is warranted in drawing from these facts the conclusion that "the average cost of masonry in a skew construction would be increased only from 3 to 4 per cent. by the heliocoidal treatment." In the first place the quantity of masonry in this arch actually requiring special cutting under the heliocoidal treatment is unusually small in proportion to the whole amount, due to the deep foundations, which extend 18 feet below the track. Furthermore it should be considered that the body of this arch is of brick, which requires no treatment, and that the special cutting in this case was really confined to the ring stone of the faces of the arch and to the sawtooth

skewbacks at the imposts, and that in cases where the entire body of the arch is composed of stone, each single piece of which would have to be cut by special patterns, the cost would be very materially increased, and with it, of course, the ratio of percentage of increase over ordinary bridge masonry.

There is but very little theoretical literature on this subject, and still less practical record of actual construction, so that every contribution adds to our knowledge and experience of a method of construction which should, and no doubt would find more extended application, if more generally understood. But those who are not familiar with the subject hesitate to undertake it, and those who are familiar with it appreciate its practical difficulties and shrink from the troublesome labors attending its execution.

A fuller understanding of the subject will overcome these difficulties and lead to a more general application of this interesting method of construction.

Mr. Crowell is entitled to the thanks of the Society, and his paper deserves the appreciation of the profession at large for shedding light upon a subject hitherto wrapped in mystery and obscurity.

Mr. Crowell—The essence of Mr. Becker's discussion is that as a theoretical exposition the paper on the Oakley Arch is worthy of the praise he bestows upon it, for which I thank him, but that from a practical standpoint the method of construction will not stand the test of economy. I should regret to have this view confirmed, because the chief object before me in preparing the paper was to establish the contrary.

By way of pointing the moral, as I suppose, Mr. Becker subjoins a statement of the expenditures at the site of the work, which, as it includes the very large items of removing and replacing the Cincinnati, Washington and Baltimore embankment, the difficult and tedious pile-foundation below water level, and the extra cost of operating the Cincinnati, Washington and Baltimore traffic due to employment of assisting engines, and is, in fact, the full account, presents a large total; but most of these items are manifestly of a character common to any type of structure that might have been used. They really have no place in the discussion, and I forbore to introduce them in the original paper. Only two items are of interest or have direct bearing, namely, first, the amount and cost of the masonry as per contract, based upon ordinary straight work, and second, the additional price paid in the form of after allowances because of the skew construction; and these, taken from Mr. Becker's statement, are as follows:

3 593 cubic yards of masonry at $10.....	$35 930
Allowances (for extra stonecutting, superior grade of stone, extra labor on centering because of maintaining traffic over head).....	1 833
	$37 763

The average cost of the masonry with the extras added amounts to $37 763 ÷ 3 593 = $10 51, which agrees with the writer's statement that the cost of the masonry was increased about 5 per cent. by the skew construction.

Predicated upon this, and bearing in mind that this extra allowance, which was made by the writer, was a liberal one, and larger than it would have been but for the necessity of maintaining the traffic overhead, I have represented that it is a reasonable expectation that the increased cost due to skew construction will not ordinarily be more than 3 or 4 per cent. greater than for the same quantity of straight work. Mr. Becker states that he does not think I am warranted in this conclusion, and in support of his position adduces the fact that the arch proper, except the ring-stones, was of brick, on which he imagines there was no extra work; as a matter of fact, however, the chief item of extra cutting was on the individual checks on the impost to receive the brick work, amounting to nearly one-third of the entire extra cost, and at least two-thirds of the cost of the check cutting could be saved in most instances by letting the impost overhang as suggested in the paper, and as would have been done here but for the excessive skew. This of itself would save 1 per cent. of the entire cost per cubic yard, and of course the shorter the impost the less the cost of this item.

Mr. Becker also calls attention to the relatively large proportion of masonry in this case, and thinks that in most cases there would be a smaller aggregate of yardage as a divisor of the extra expense; this point, which has already been brought out in the verbal discussion, is well taken, but there is, of course, a relation between the height of the arch and the height and extent of the wing walls, so that the difference as between the quantities in different structures would be largely in the depth of foundations, and so this case is a favorable one for the percentage in that respect. But on the other hand, the writer is satisfied that the extra labor in cutting the ring stones should not be nearly so great as was allowed for, and in fact beyond the first cost of stone cut to waste, due to the shape, there should be no extra price for skew-cutting at all. The tool work on the soffit and face is exactly the same for the same sized stones in skew as in straight work, and the remaining work, on the winding beds, differs only in the use of the "twist rules" instead of the straight edges. It is entirely a question of "know how," and while contractors as a class may not possess the knowledge, there is no actual difficulty in teaching the stonecutters, and if more frequent opportunities were given, there would be more facility.

The less the degree of skew, of course, the less twist in the stones and the less waste, and this gives another element in forming the reasonable expectation.

But the argument after all is—and this is what was claimed—that in most cases the economy in quantity of material will more than offset the cost of the skew-cutting.

Mr. Becker makes another criticism to the effect that he thinks the alternate trestle system proved more costly than a truss system of temporary support spanning the entire opening would have done, a conclusion to which he is led, I suppose, by contemplating the cost of the extra motive power paid to the Cincinnati, Washington and Baltimore Company, amounting in his schedule to $4 138 07. This has no bearing on the skew construction, and is of no great importance, because the choice of false works in any case would depend on purely local considerations. But as the point is raised, I will say that in my visits to the work during its progress I gave that question much consideration, and am of the opinion now that if again confronted with a similar problem, I would feel safe in using the trestle system, and would not be justified in adding to the natural difficulties the effect of the heavy concentration of weight which the truss span would entail in dangerous proximity to the work on the foundations, even if a choice was possible. There might be cases where the foundation question could be eliminated, and where undoubtedly a truss system of false works would be better.

And the fact remains, like the celebrated excuse of the French *maire*, that in this case there was not sufficient head room to work the pile-driver under the track, despite Mr. Becker's ingenious theory based on the length of the bearing piles after they were cut off. I will add that the trestle that was used was quite as substantial as any truss, and heavy freight trains, as it happened, were run sometimes over it at very high speed, in defiance of instructions. The trains were slowed as a wise precaution, and the same regard for the safety of the traffic would have called for the same regulation in running over any false works. The trestle did not retard the progress of the work, and the latter was the measure of the item of motive power.

PLATE XXXVI. REMOVING THE EMBANKMENT—NORTH END. Vol. XXIII, p. 192.

PLATE XXXVII. ALTERNATE TRESTLE IN USE. Vol. XXIII, p. 192

PLATE XXXVIII. IMPOST AND CENTERING. Vol. XXIII, p. 192.

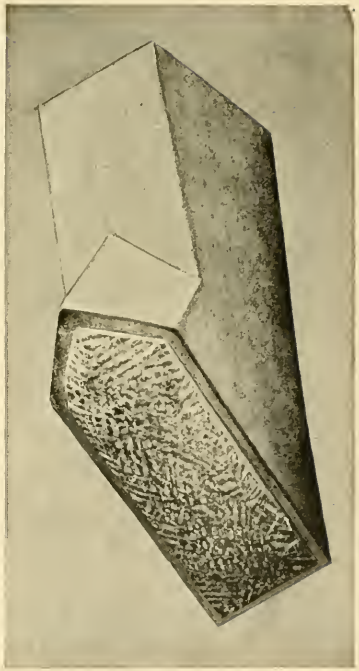
PLATE XXXIX. TURNING THE NORTH FACE. Vol. XXIII, p. 192.

PLATE XL. FINAL STAGE. FILLING TO GRADE. Vol. XXIII, p. 192.

PLATE XLI. BUTTRESSED LOCKING. Vol. XXIII, p. 192.

PLATE XLII. THE OAKLEY ARCH—NORTH FACE. Vol. XXIII. p. 192.

PLATE XLIII. VIEW IN THE STONE YARD. Vol. XXIII, p. 192.



A SPECIMEN STONE.

Foldouts
Updated 3/18/2004

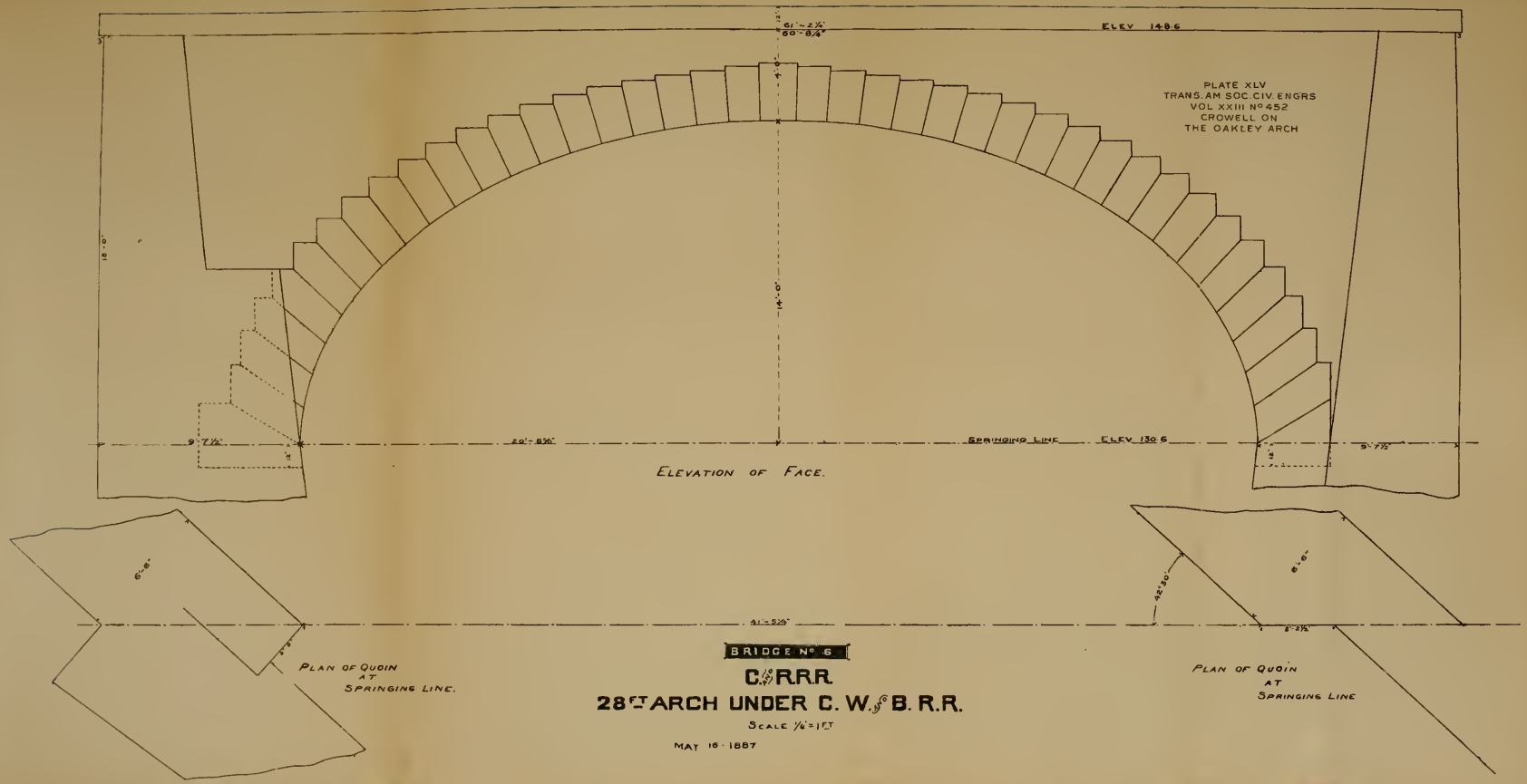


PLATE XLV
TRANS. AM SOC. CIV. ENGRS
VOL XXIII Nº 452
CROWELL ON
THE OAKLEY ARCH
ELEV 148.6
61'-2¼"
60'-8¼"
4'-0"
14'-0"
18'-0"
9'-7½"
20'-8⅝"
SPRINGING LINE
ELEV 130.6
9'-7½"
ELEVATION OF FACE.
6'-6"
41'-5½"
42°30'
8'-6"
PLAN OF QUOIN
AT
SPRINGING LINE.
PLAN OF QUOIN
AT
SPRINGING LINE
BRIDGE Nº 6
C. & R.R.R.
28 FT ARCH UNDER C. W. & B. R.R.
SCALE ¼" = 1 FT
MAY 16 - 1887

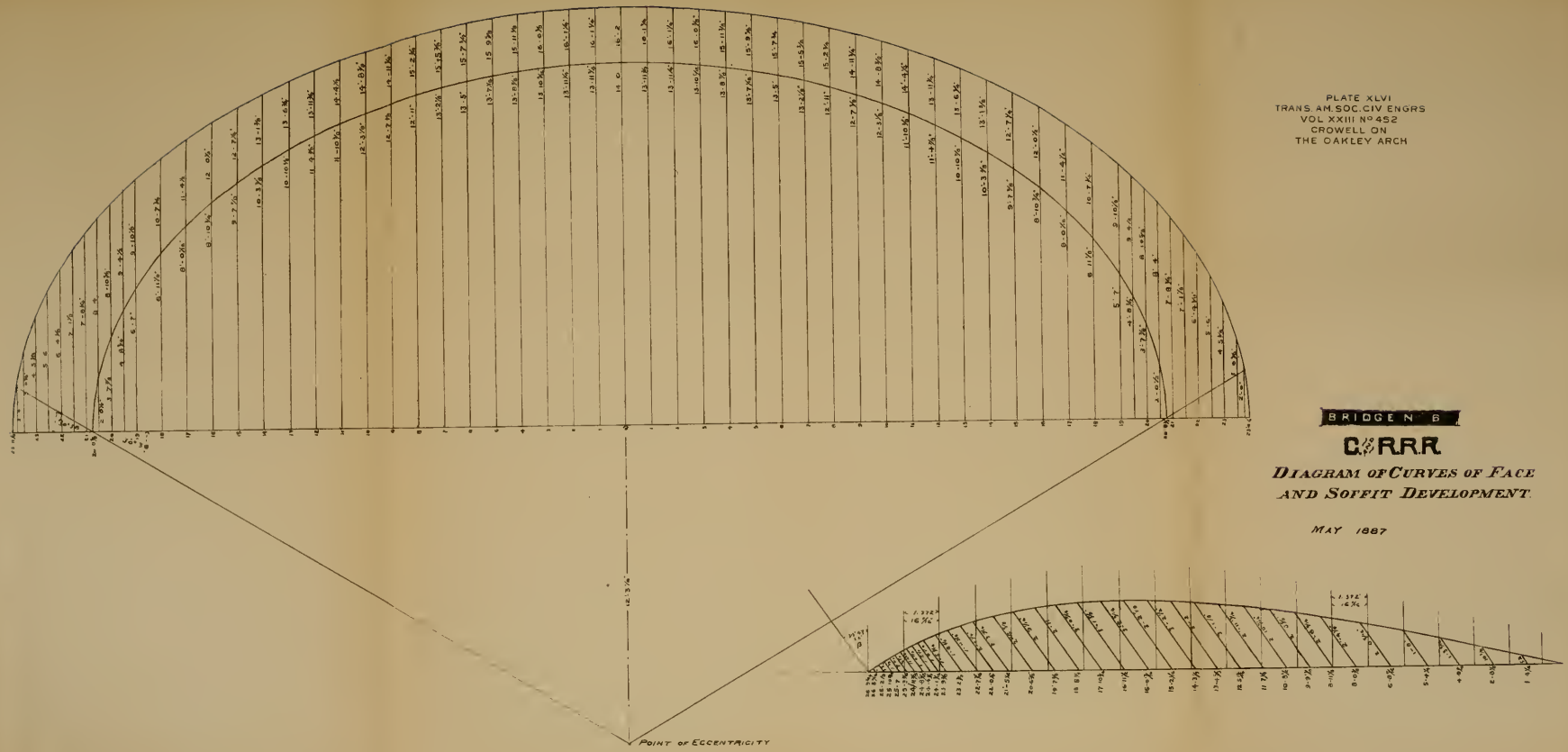


PLATE XLVI
TRANS. AM. SOC. CIV ENGRS
VOL XXIII Nº 452
CROWELL ON
THE OAKLEY ARCH

BRIDGE N 6

C. & R.R.R.

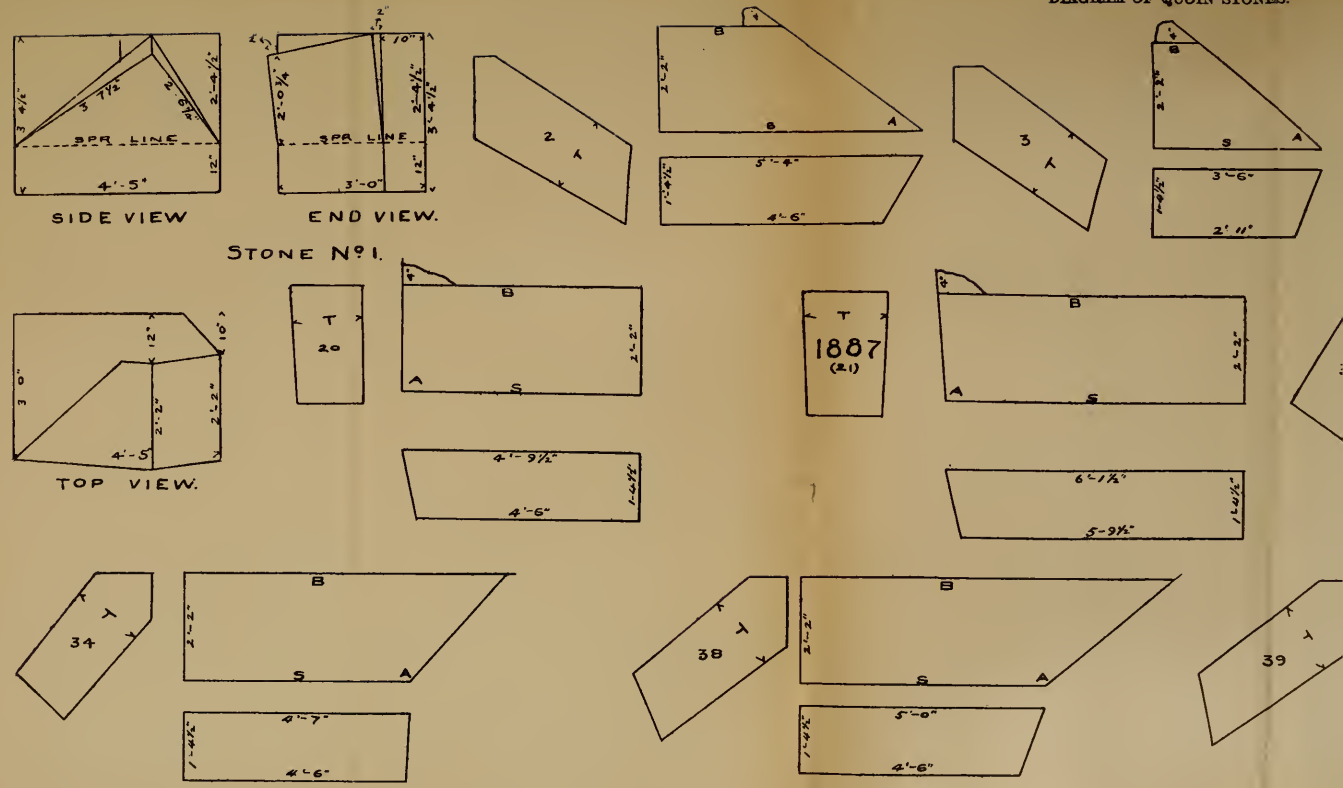
DIAGRAM OF CURVES OF FACE AND SOFFIT DEVELOPMENT

MAY 1887

QUARRY DIMENSIONS.

STONES.					
Nº	BED	ANGLE	SOFFIT	BACK.	THICKNESS
2	LEFT	37°56'	5'-4"	2'-6 5/8"	2-0"
3	"	40°31'	3-6	0'-11 1/2	1-11"
4	"	41°54'	5-0	2-7	2-0"
5	"	44°41'	3-6	1-3 3/4	1-11
6	"	46°14'	4-9	2-8 1/8	2-0
7	"	49°14'	3-6	1-7 5/8	1-11
8	"	50°58'	4-7	2-9 7/8	1-11
9	"	54°09'	3-6	1-11 1/4	1-11
10	RIGHT	58°49'	4-7	3-3 1/4	1-11
11	"	62°18'	3-6	2-4 3/8	1-11
12	"	64°40'	4-8	3-7 3/4	1-11
13	"	68°13'	3-6	2-7 5/8	1-11
14	"	70°39'	4-9	3-11 7/8	2-0
15	"	74°30'	3-6	2-10 3/4	1-11
16	"	77°05'	4-9 1/4	4-3 1/4	2-0
17	"	81°04'	3-6	3-1 7/8	1-11
18	"	83°47'	4-9 1/2	4-6 5/8	2-0
19	"	87°52'	3-6	3-5	1-11
20	"	90°28'	4-9 1/2	4-9 3/4	2-0
K 21	"	95°22'	6-1 1/2	6-4	2-1
22	"	98°08'	4-9 1/2	5-2 3/8	2-0
23	"	100°53'	3-6	3-11	1-11
24	"	104°56'	4-9 1/2	5-4 1/2	2-0
25	"	107°35'	3-6	4-2 1/4	1-11
26	"	111°30'	4-9 1/4	5-7 1/2	2-0
27	"	114°01'	3-6	4-5 5/8	-11
28	"	117°47'	4-9	5-10 3/4	2-1
29	"	120°00'	3-6	4-9	2-0
30	"	123°42'	4-8	6-1 3/8	2-1
31	"	125°51'	3-6	5-0 3/4	2-0
32	"	129°14'	4-7	6-1 3/4	2-1
33	LEFT	128°34'	3-6	5-2 3/4	2-0
34	"	131°51'	4-7	6-6 1/4	2-1
35	"	133°42'	3-6	5-6 7/8	2-0
36	"	136°46'	4-9	7-0 5/8	2-2
37	"	138°26'	3-6	5-11 3/8	2-1
38	"	141°19'	5-0	7-8 1/2	2-2
39	"	142°46'	3-6	6-4 1/4	2-1
Nos 2 TO 23 INCL 2'-6" WIDE					
" 24 " 39 " 2'-2" WIDE.					

DIAGRAM OF QUOIN STONES.



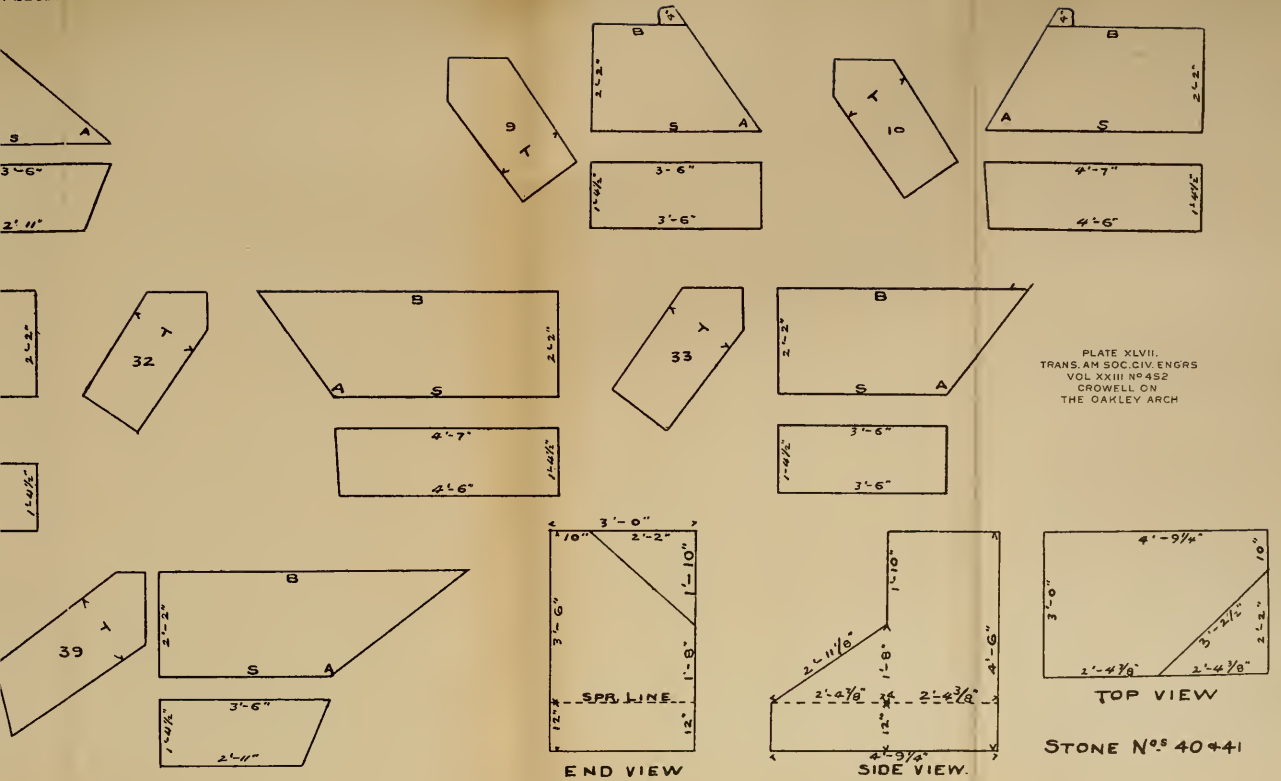


PLATE XLVII.
TRANS. AM SOC. CIV. ENGRS
VOL XXIII No 452
CROWELL ON
THE OAKLEY ARCH

STONE Nos 40 & 41

HORIZONTAL PROJECTION OF SPIRAL.

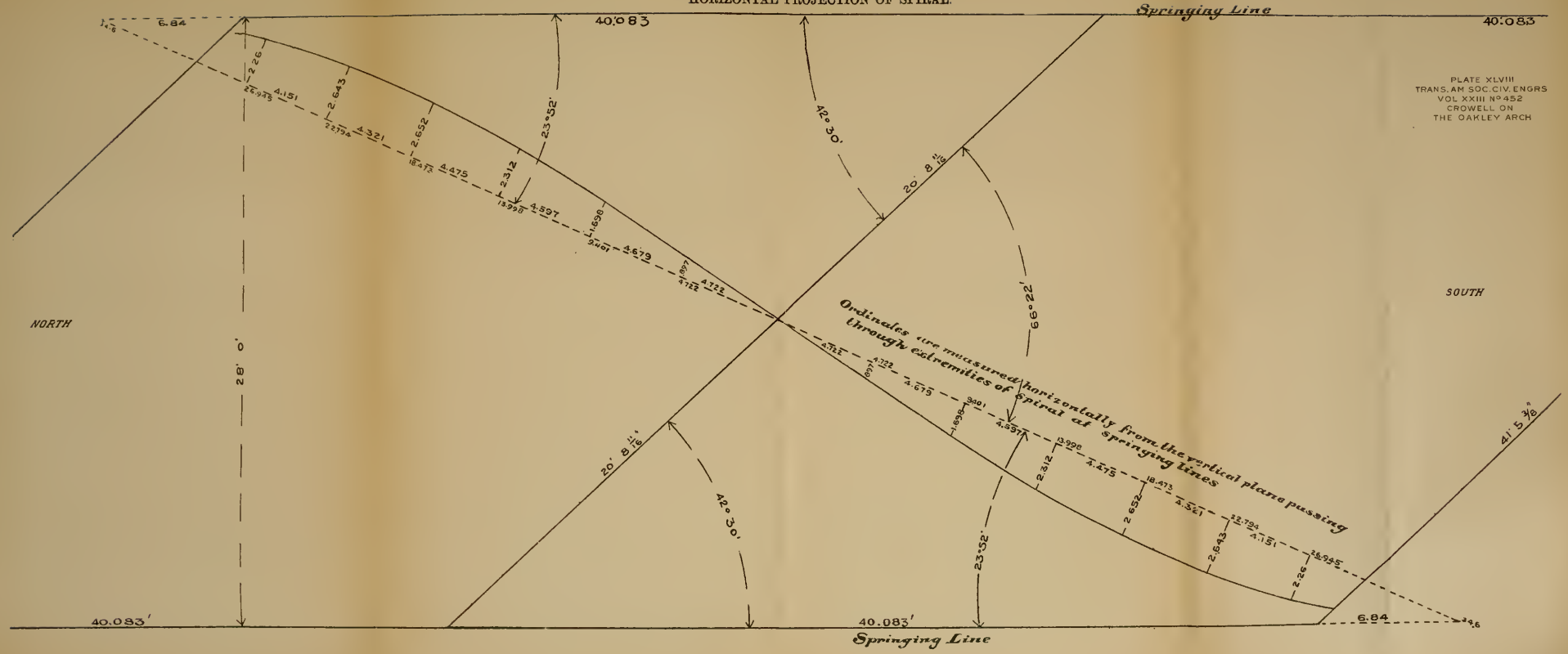


PLATE XLVIII
TRANS. AM SOC. CIV. ENGRS
VOL XXIII Nº 452
CROWELL ON
THE OAKLEY ARCH

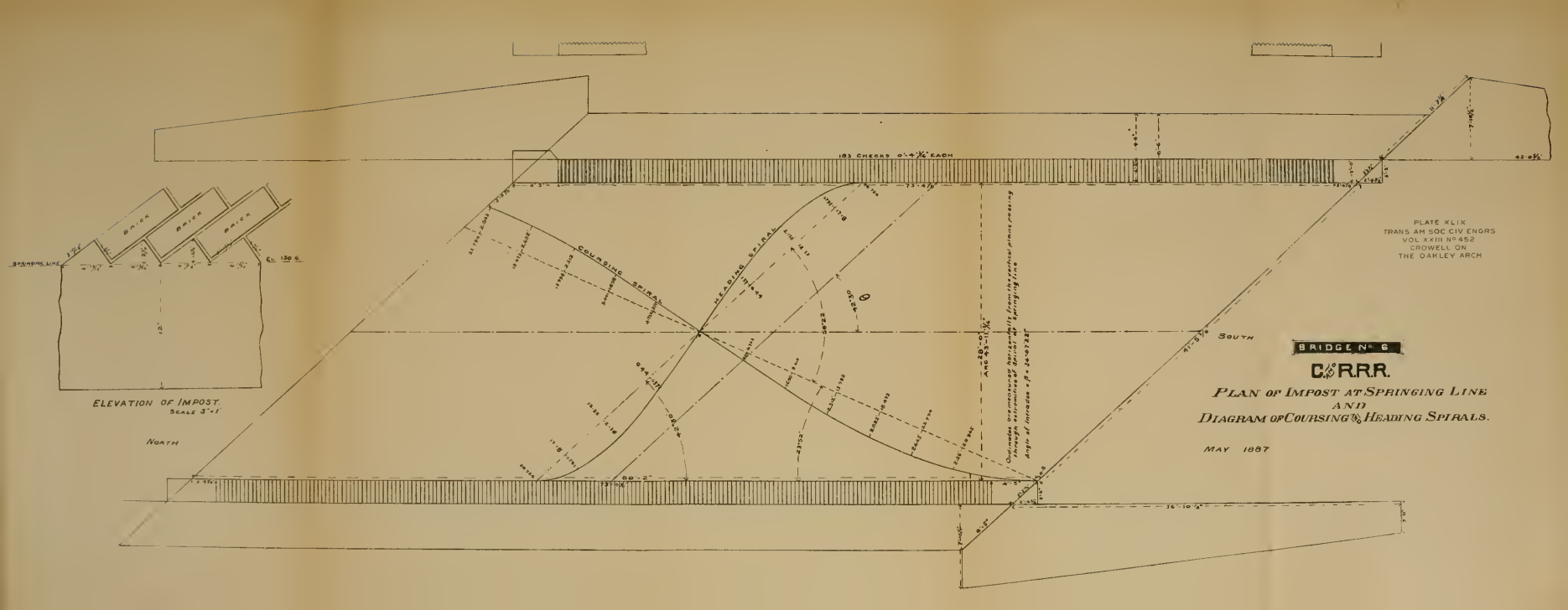


PLATE XLIX
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII. No. 452
CROWELL ON
THE OAKLEY ARCH
BRIDGE No. 6
C. & O. R.R.R.
PLAN OF IMPOST AT SPRINGING LINE
AND
DIAGRAM OF COURSING & HEADING SPIRALS.
MAY 1887
ELEVATION OF IMPOST.
SCALE 3"=1'
NORTH
SOUTH
183 CHECKS 0'-4 3/16" EACH
COURSING SPIRAL
HEADING SPIRAL
BRICK
BRICK
BRICK
SPRINGING LINE
EL. 130.6

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII. N° 452.
CROWELL ON
THE OAKLEY ARCH.

SKETCH OF LAGGING
WITH
SPIRALS MARKED FOR BRICK WORK

PLATE LI
TRANS. AM SOC. CIV ENGRS
VOL XXIII Nº 452
CROWELL ON
THE OAKLEY ARCH.

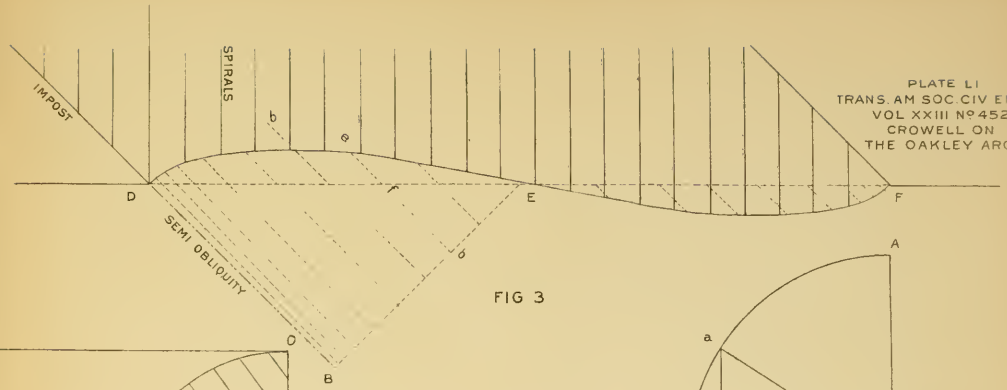


FIG 3

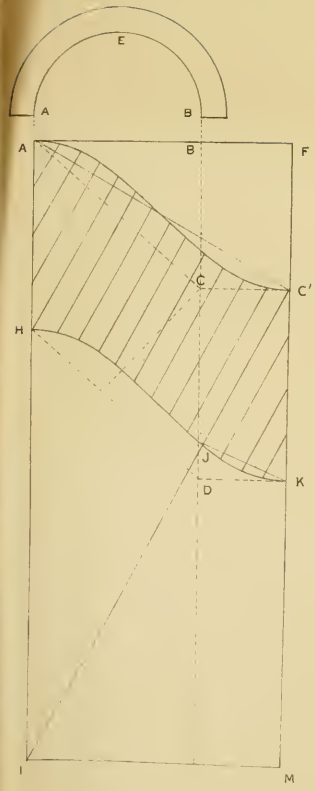


FIG 1

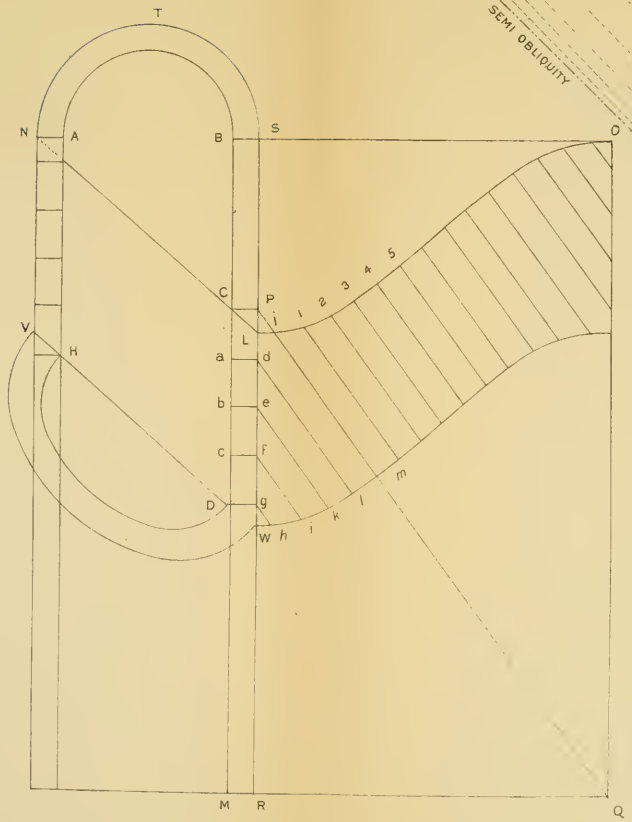


FIG. 2

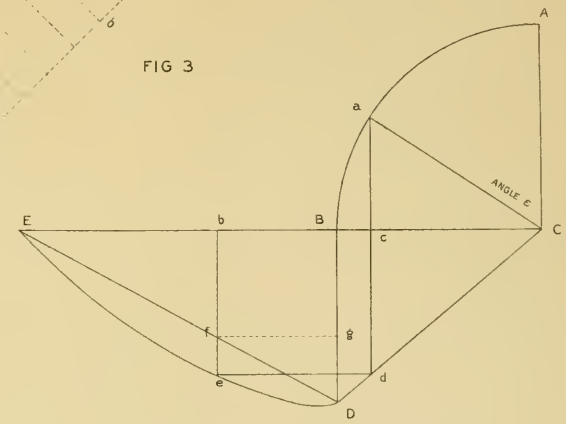


FIG. 4

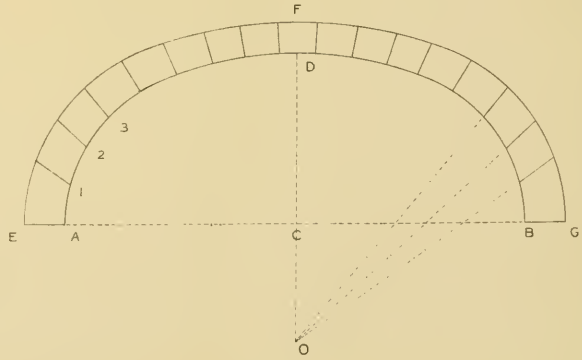


FIG. 5

PLATE LII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII Nº 452.
CROWELL ON
THE OAKLEY ARCH.

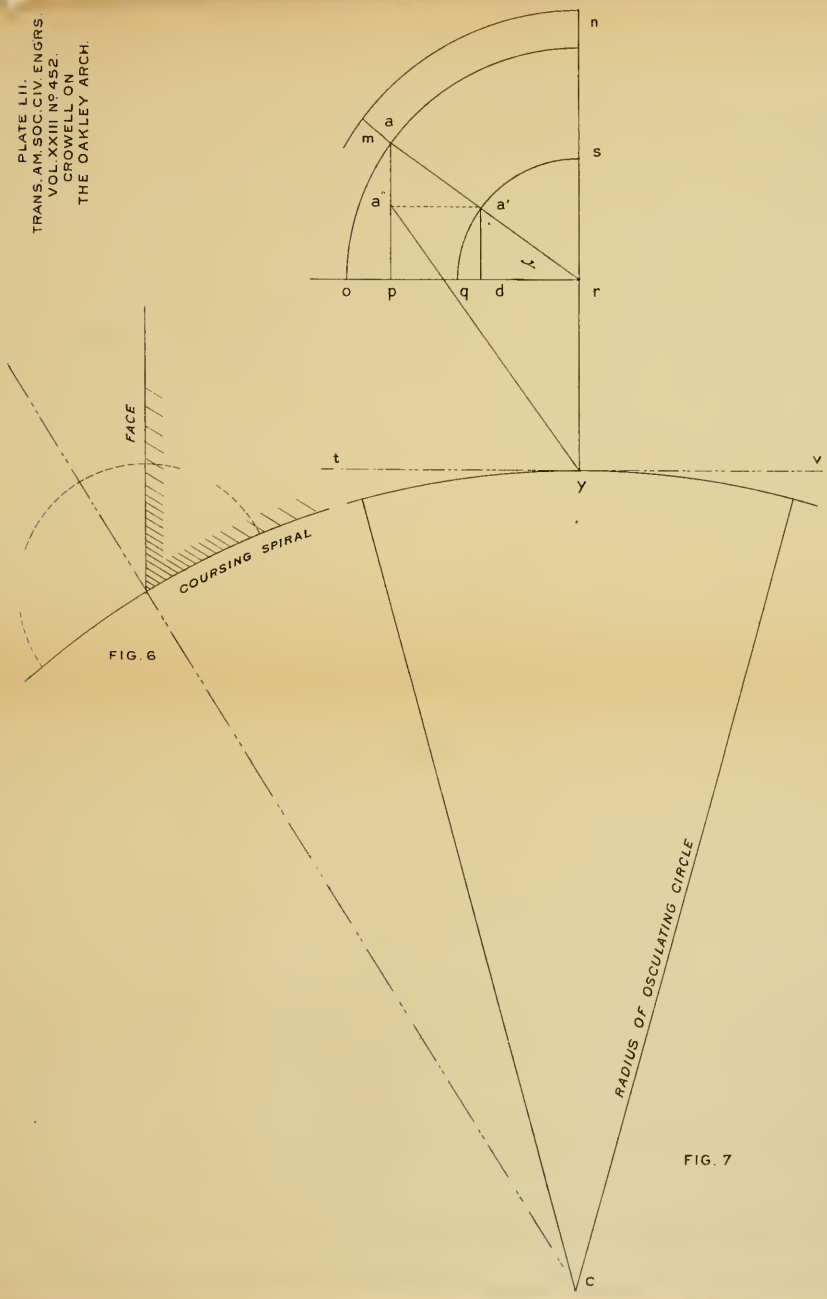


FIG. 6

FIG. 7

PLATE LIII
TRANS. AM. SOC. CIV. ENGRS
VOL XXIII Nº 452.
CROWELL ON
THE OAKLEY ARCH

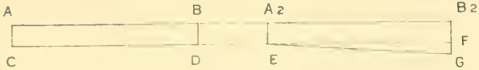


FIG. 8

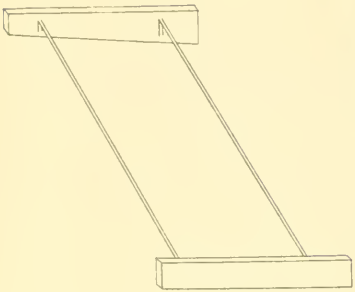


FIG. 9

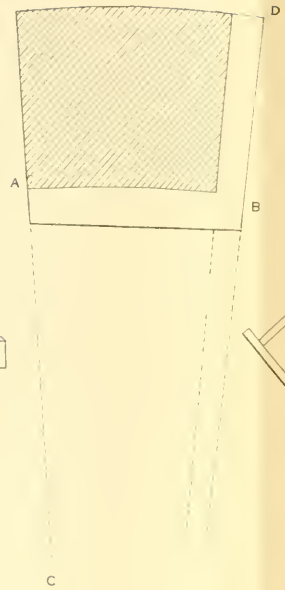


FIG 11

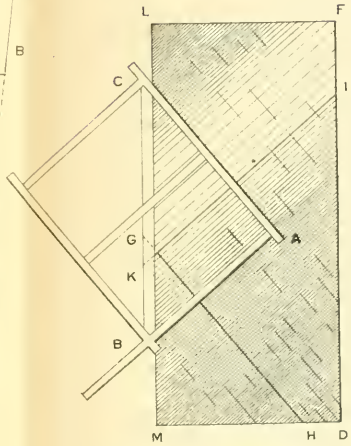


FIG 13

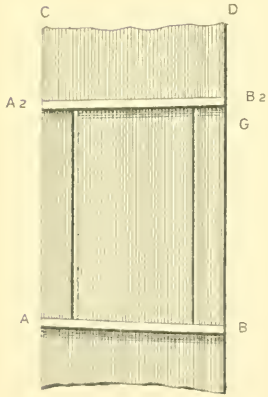


FIG 10

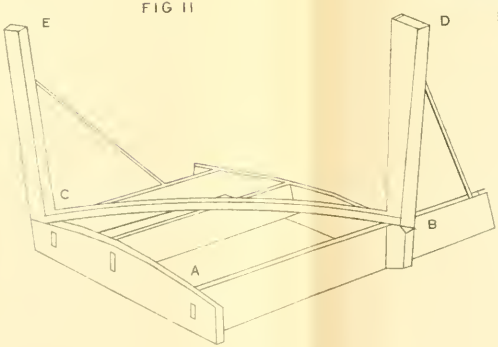


FIG. 12

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453.

Vol. XXIII.—October, 1890.

COMPARATIVE TESTS OF AN ELECTRIC MOTOR AND A STEAM LOCOMOTIVE ON THE MANHATTAN (ELEVATED) RAILWAY, NEW YORK.

By LINCOLN MOSS, C. E.

WITH DISCUSSION.

In February, 1889, the writer was detailed from the Chief Engineer's Department by the General Manager, Colonel F. K. Hain, M. Am. Soc. C. E., to conduct tests to determine the efficiency of an experimental "direct" electric system of motive power, as compared with the present steam locomotive performance. The system was one of three which had been tried at various times on the elevated roads, and the only one which had been pushed far enough to be recognized as a possible competitor of the steam locomotive, and much credit is due to the inventor's skill and persistence.

The section of road chosen for the experiment was 1.76 miles long, from 12th to 51st street on the Ninth avenue line. A profile of this section is shown on Plate LVI, giving the grades, stations and streets. This section is on the oldest and lightest of the four lines operated by the Manhattan Railway Company; the traffic is comparatively light, as is also the rolling stock; the cars weigh twelve tons each, empty, and the locomotives 16 to 19 tons, in working order. The track was in good condition, but was not up to the present standard, as the replacing of the old 50-pound rails and 5-inch ties with the standard 90-pound rails and 6 x 8-inch ties would have interfered with the electrical conductors.

The trial trains were made up of empty cars of the new standard type, which weigh 29 500 pounds each, empty; at first two were pulled, then three, and finally four; these four empty cars weighed a trifle less than the four-car trains, loaded, which are run on this line in regular service. The trial cars have each two four-wheeled trucks of the swing bolster pattern, with four wheels, 30 inches diameter, which are of the steel-tired Allen paper variety.

After the experiments with the electric system were concluded, a steam locomotive was detailed from regular service to pull the same train over the same section and under the same conditions, as far as possible, to determine the relative efficiency.

Description of the Electric Plant.

The generating machinery was located at the south end of the section in a side street, and about 400 feet west of the railway structure.

The boilers were of the ordinary horizontal tubular pattern, set in brickwork on the return system, rated at 100 horse-power each, and furnished steam at a pressure of 80 to 90 pounds per square inch; two boilers were used in the lighter runs, and three subsequently.

The engine was a horizontal "Wright's automatic cut-off," with a single cylinder, 22 inches bore by 42 inches stroke, and worked non-condensing; the 11-inch crank-shaft carried a fly-wheel 16 feet diameter, 2 feet face and 1¼-inch rim, which revolved, in the later experiments, from ninety to one hundred times per minute, depending on the resistance, giving a piston speed of 630 to 700 feet per minute, and a velocity of belt which ran from the face of the fly-wheel to the main shaft pulley, of 4 524 to 5 026 feet per minute. This engine was in charge of experts, and was fined down to practically perfect adjustment, showing but 5 per cent.

friction. The greatest indicated horse-power found during the trials was 400.

There were four generating dynamos, nominally 50 horse-power each ; they could be worked in parallel or in series by means of a switch, and were driven by belts and wooden pulleys from the main line shaft. There was a small 4-horse-power dynamo in addition, which supplied current for the engine room incandescent lights.

The current was conveyed from the dynamos to the railway structure conductors by two insulated copper cables strung on poles ; at the structure the cables were divided to connect with the three positive conductors on the east, center and west tracks.

The positive conductors, Plate LX, were of very fine copper, ⅝-inch round rods, bolted to the wooden guard timbers by insulated clamps.

The negative conductors were made up of the east line rails of each track ; they were connected by ordinary angle splice bars and bolts, and also were connected electrically by copper wires at each joint, riveted in the flanges.

The positive conductors of the east and west tracks were cross connected at intervals of about 200 feet.

The circuit was closed through the motor on the track by a device which had copper leaved brushes which slid over the positive conductors, and the current, after passing through the motor dynamo and its connections, was returned through the driving wheels into the rails, passing thence back to the generating dynamos. Another copper brush was provided on the opposite side of the motor to bridge gaps in the positive conductors on the switches.

Sketches of the motor are shown on Plates LVI and LX; the four driving wheels were 48 inches diameter, 6 feet between centers, and coupled to obtain the maximum tractive force ; on the rear axle were two carefully cut steel gears, driven by two steel pinions keyed on the shaft of the dynamo ; on this shaft were the armature and commutator revolving between the field magnets ; the brushes were movable and could be reversed to change the direction of rotation, but not when under headway; the brake shown was electro-magnetic, but was more effective when applied by the hand wheel and screw; the motor could be cut out of the circuit, but this was not done for fear of injury to the mechanism. The exact weight of this motor was unknown ; it was known to be 10 tons,

and probably weighed more. The motor dynamo was said to possess 128 horse-power.*

The middle handle shown at the front of the machine was used at first to vary the speed of the motor by raising or lowering the resistance of the field magnet circuit, or the strength of the field, but in the later and heavier experiments additional resistance was introduced. The driving wheels were allowed to play in the pedestals a trifle by means of rubber springs, and the teeth of the pinions consequently moved up and down between the teeth of the gear wheels when vibration took place.

One purpose of returning the current through the driving wheels into the rails was to increase the adhesion or frictional co-efficient between the wheels and the rail. While the writer believes this claim has foundation, it could not be proved, as the motor did not possess sufficient power to slip its own wheels against the friction due merely to the actual weight upon the wheels. The maximum pull in a test to determine this, if possible, was 4 500 pounds for an instant, on a dry track, which is often sanded at that point by the engines hauling express trains. This pull of 4 500 pounds would give a co-efficient of friction for 10 tons concentrated on the drivers of .225, while a co-efficient of .35 is not unusual at such a place in locomotive practice. The motor was equipped with sand pipes at each end, which are not shown, nor are the resistances, switches and various wire connections shown.

Plate LVI is a graphic record of the heaviest test attempted, made on the 15th of April, 1889, at 10 P.M. There was no appreciable wind blowing to complicate the question of train resistance, and the weather and track were dry, the latter conditions being especially favorable for good electrical results.

The train consisted of four empty cars, weighing 29 500 pounds each as before described. This train weighed somewhat less than a fairly loaded four-car train, such as is run on this line during the busy hours of the day, and the test was an attempt to imitate the run of such a train, stopping at all the stations on the way. In the absence of any other brake aside from the motor brake, all the car brakes were manned to make the stops quickly.

The first profile shows the track with its grades, the stations and streets.

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* This is the builder's figure as the result of a Prony Brake test.

The second diagram shows the indicated horse-power of the driving engine, which is the prime mover to be considered, and which propels the train through the medium of the electric current. In all cases 4 horse-power has been deducted from the cards, for the engine room lights. The automatic cut-off of the engine was quite sensitive, and there was considerable fluctuation in the power called forth; for this reason the writer took the indicator cards as rapidly as possible, to secure the greatest possible number of readings, and averaged a card every 16 seconds throughout the run. An assistant recorded the revolutions at the instant of tracing the diagram, with a tested tachometer; another assistant timed each card accurately, so that knowing the time of the arrival of the train at the various points along the run, the card may be placed very accurately at the point where the train was at the same time.

The third profile shows the net or effective horse-power exerted at the draw-bar of the motor and is the measure of the power used to propel the train; it is also the measure of the fraction of the power developed by the prime mover actually recovered for useful work at the draw-bar of the motor. The average power generated was 176.8 horse-power, while the amount recovered on the track was 26.7, making the average efficiency .151 and the average loss $84\frac{9}{10}$ per cent.

The coupling link between the motor and train was a Shaw hydraulic dynamometer (Plate LV), with a 12-inch gauge graduated in hundreds of pounds. Just prior to this test, the dynamometer was sent to the maker's shop in Philadelphia, where it was put in thorough repair and tested, and the pointer adjusted and set by an expert in the employ of the electric company. The readings were called out by one observer and recorded by another who also noted the position of the train at the same time.

The net horse-power was calculated by formula.

$$\frac{\text{Pull of Motor in Pounds by Velocity of Train, Feet per Minute}}{33\,000.}$$

$$= \text{Net Horse-Power.}$$

The fourth diagram records the "pull" of the motor on the train at the draw-bar in pounds, as observed.

The fifth diagram shows the velocity of the train in feet per second,

recorded by two observers on board the train, one of whom would call out as the train reached the center of an intersecting street or a station, while the other would carefully note and record the time on a prepared form. The times of arrival and departure from stations were also noted. The various distances having been carefully measured, the velocity of the train was obtained by dividing the length of a given block by the number of seconds consumed in passing it, for the velocity in feet per second.

The average net speed, all stoppages deducted, was 9.4 miles per hour; the average service net speed in the regular operation of this section is 15 miles per hour.

Referring to Diagram 2, Plate LVI, it will be noticed that the power expended by the prime mover never went back to zero, even when the train and motor were standing at a station, at which time there was a constant average expenditure of 38.2 horse-power. Of this about 4 horse-power is lost in the engine and shafting, 5 horse-power of the remainder is lost in the dynamos, leaving 29.1 horse-power lost in the line.

The greatest power of the engine was exerted when the train was being started. For this reason it was claimed at one time that better running time could be made with electric motors than with steam locomotives, which exert the least horse-power in starting, although the steam is admitted to the cylinders at nearly full stroke. The fallacy of this belief becomes apparent by referring to diagrams 3, 4, and 5, Plate LVI. The net horse-power actually exerted upon the train is least when the train is being started, the dynamometer shows a low initial pull, with a characteristic falling back, and the speed line invariably shows a hollow curve of acceleration in place of the full curve which would indicate a rapid acceleration of speed, and a correspondingly great exertion of power against the resistance of the train. From a consideration of these facts, we may inquire, where was the great loss in starting, notably at 30th street station, where the driving engine indicated 395 horse-power, while at the same time, but 7.6 horse-power was being exerted to pull the train, showing less than 2 per cent. of the power of the engine transmitted to the train at that instant? The chief losses are due to several facts in the operation of electric motors.

First.—The efficiency of a motor depends largely upon the development of the counter *E. M. F.*, clearly shown in this case, the efficiency

increasing almost directly as the increase of train velocity. The heavier the load and the slower the speed, the poorer the economy. The motor, ever changing in speed and requirements, could not maintain the counter current at the most economical point or intensity, as in a stationary motor running with a nearly constant current and potential. When the motor is just beginning to move, this counter current is small and increases as the armature revolves faster. The effect of the increase of this counter electro-motive force is to diminish both the volts and the ampères, as well as the current waste, which is greatest when the motor armature is just beginning to move. The counter current could be increased till it equaled the driving force. The golden mean can only be obtained in a motor running at a fixed speed, with relation to the generator.

Second.—The economy is greatest, other things being equal, when the load is lightest, and the counter current is largest.

Third.—When a motor is being started, there is no counter current, and it is necessary to increase the resistance by rheostats, shunts, or resistance coils, or by varying the strength of the field, and these methods increase the current waste in accordance with Joule's law of current waste ($C^2 \times R =$ current waste) "C" being the current in ampères, and "R" being the resistance in ohms. In a system where a stationary motor is to be driven at a constant speed under an unchanging load, it is possible to adjust the various relations of size, speed, counter electro-motive force, etc., so as to reclaim a very good proportion of the force generated in the prime mover. As much as 60 or 70 per cent. has been claimed in such instances, but in an electric locomotive the conditions are the very worst for economical results, by reason of the ever-changing speeds and loads and the complication of conditions; and the heavier the load to be handled and the greater the speed demanded, the more difficult and complicated the problem becomes. The most safe and reliable method of regulation is doubtless the rheostat or resistance coil. Some have used shifting commutator brushes; others have used shunts to vary the strength of the magnetic field, and in the case before us various devices were used experimentally, and the best and most reliable only were adopted.

Devices that would answer for a 10 horse-power electric street car at ordinary speeds would not answer the purpose on a 128 horse-power motor handling far greater loads with much larger variation in speeds.

One effect of introducing a resistance into the circuit is to increase the horse-power, the formula for which is:

$$\frac{E \times C}{746} = \frac{\text{volts} \times \text{ampères}}{746} = \text{horse-power.}$$

The volts measuring the pressure, and the ampères the current, the current being supplied by the generators at nearly constant pressure or voltage, it becomes necessary to increase the power in starting, either by increasing E or C. It is also necessary to use a resistance to prevent burning out of the commutators and connections. At the instant of starting, much current is being used, a great deal of heat is developed by current waste and resistance, and as the counter current is practically 0, it is imperative, both for economy and safety, to increase the power by increasing the pressure (E) instead of the current (C), which is done according to Ohms' law, $E = C \times R$, by introducing a resistance into the circuit, electro-motive force (volts) = current [ampères] × resistance (Ohms).

$$E = C \times R$$

The action of an electric motor bears some resemblance to that of a steam locomotive. In the latter, in starting, the distribution of the steam is the most wasteful, and the average pressure on the pistons is the greatest, when "notched up" so as to expand the steam more economically, the economy is very much improved, and the average pressure falls comparatively low, but although low, it is applied so much more rapidly, that a greater horse-power is developed. So in an electric locomotive, the electrical pressure is greatest when starting, because of the "resistance" introduced, and the current used is greatest, owing to the absence of counter-electro-motive force; but as the speed increases and the resistance is cut out and the counter current increases, the electrical pressure becomes less, and also the current used, for it is evident that if the speed of the motor was increased enough the new or counter current, generated by the driven motor acting as a generator, would equal the current derived from the generating dynamo, and no work would be done by the motor; so that when the motor is working rapidly and the impulses are of low pressure and quantity, the efficiency is the greatest, although it seems like a strange kind of efficiency which depends upon the development of an opposing force.

Of course the heavier the load, or the steeper the grade to ascend, the greater is the "resistance" required in order to increase the horse-

power, and as the speed of ascent on a grade is below the normal, the counter current is small and the current waste is greater, as more current is used, so that we might expect to find at 30th street, which is at the foot of the $1\frac{86}{100}$ per cent. grade, the greatest waste of power and the least efficiency. On this occasion the rear end of the train was on the down grade. When the current was switched into the dynamo to start the train, the resistance was immediately felt at the power station, the dynamos and driving engine slackened their speed about 10 per cent., which allowed the arms and weights of the centrifugal governor to fall and increase the period of admission in the steam engine cylinder. The greater the load the lower would the governor arms drop, so that in starting out from 30th street the initial card shows the steam admitted for the full stroke. On one occasion, at this point, the engine worked so vigorously against the resistance as to burn out the brushes of one dynamo. Of course in handling heavy loads, when much current is wasted, it shows itself in the form of heat. On another occasion, in running with an experimental combination, sufficient heat was liberated to set fire to the insulation and woodwork in the motor.

Some persons claim a restoration of power on the down grades, upon the theory that the armature in the motor, propelled by gravity, will generate a current which will be restored to the line. While this should be so theoretically, provided the armature revolved rapidly enough, it will suffice to state that a careful comparison of the light cards taken while the train was running down the long 1 per cent. grade from 34th to 42d street, by gravity, with the cards taken when the train was not in motion at all, and the generating engine was running "light," shows no perceptible diminution.

At first, in these experiments a current of low potential was used, but as the loads were increased and the usual stops and starts made with the load, it became necessary to increase the power. This was done by increasing the speed of the engine from 80 to 100 revolutions per minute, the effect of which was to increase the potential by revolving the dynamo armatures faster. While the current used was not deadly, it was strong enough to knock a man off his feet, and as it was exposed not only in the conductors but also in the wheels, side-rods and other mechanism about the motor, in practice, it would have been imperative to use a pressure in the neighborhood of that prescribed by the London Board of Trade, 300 volts.

To obtain a given horse-power by a low potential current would mean the use of more current and larger conductors for the horse-power $= \frac{\text{volts} \times \text{ampères}}{746}$. If we reduce the volts, the ampères must be increased, and the ampères are the product of the current divided by the resistance or $C = \frac{E}{R}$. We should also recollect that the current waste is equal to the square of the current multiplied by the resistance, or $C^2 \times R$, so that the economy of high tension currents is apparent, a point well understood by electric light companies. Of course the higher the *E. M. F.*, the better must the insulation be, just as high pressure engines and boilers require better construction. Had the voltage been kept at the original low *E. M. F.*, the economy would not have been as good as that shown on the diagrams.

High speed of armature is an important requisite, and as yet no means have been found practical to apply the motor armature to the driving axle direct. Direct action and elimination of gearing and intermediate contrivances is the best for all engines, whether stationary, locomotive or marine, and the electric motor was undoubtedly at a disadvantage in being compelled to use gearing to reduce the speed; especially should we expect quite a loss in friction, because the pitch lines of the gear wheels and pinions were constantly varying as the axles moved up or down in the pedestals. This motor originally was built with *V* friction gearing, which was found to be unreliable and useless owing to the oscillations. The loss due to varying counter current and current waste is of course common to all electric motors, from whatever source the current is derived. Some inventions have been made to obviate this great source of waste, by keeping a constant rate of armature speed and using variable intermediate gearing, and other inventions have been made with this purpose in view; but spur gearing remains still the standard. The objectionable use of resistances is done away with on storage battery motors by grouping the cells which represent about 2 horse-power each, into series, so that the operator may switch only one set or all into the circuit, and thus obtain a current of variable intensity and pressure. If but one motor on a direct system circuit was in use, it would be possible to apply the same principle to the direct system, but in practice there would be a number of motors on a section at once. Some would be just starting and calling

for much current and high potential; others might be in full speed and calling for a current of lower pressure and quantity, so that it would be impossible to suit all the varying conditions at once. A great many trials were made under varying conditions, and one interesting fact among others developed, is, that the economy with light trains at higher speeds becomes poorer, apparently contrary to the general principle that the economy of stationary electric motors increases with the speed and the reduction of the load, and many believed that a better showing would be made with such tests. It is true that tests made with light trains of two or three empty cars run at a high speed, but making no stops, showed much better results, but such conditions are not at all practical in operating elevated or other railways. It is true that a lightly loaded stationary motor, running at a nearly constant high rate of speed, will show maximum economy, but in the case before us, where the motor was changing its speed continually and stopping and starting, the rule will not apply, To demonstrate the fact, and also to form an estimate of the probable power required at a higher speed than was attained in the trials, the writer has prepared the diagram shown on Plate LX. The base line is laid off in spaces representing speed in miles per hour, from 8 to 15; then taking a number of reliable tests at different speeds and with differing loads, and dividing the total weight of the train in tons into the average indicated horse-power, we obtain the horse-power per ton of train, expended at the various speeds. These results have been plotted, and the ordinates connected by a line which forms a tolerably regular curve, considering the various little changes made in the motor at different times, and the varying atmospheric conditions. The four right-hand ordinates were obtained from tests made entirely by the experts of the electric company. Two were made with two car trains, and two with three car trains (all empty). The other ordinates were taken from tests with four-car empty trains, made by my assistants and self. The lower lines show the net horse-power required at the various speeds per ton of train, as determined by the locomotive tests, and also the required indicated horse-power per ton of entire train.

The extreme right-hand ordinate was figured from a test with two cars weighing 59 000 pounds together. The average rate of speed was 12 miles per hour net, and no stop was made at the most difficult point, 30th street; yet the average indicated horse-power is more than that found

with a train of double its weight, and running only 2.6 miles per hour slower.

It being evident that the electric motor had not sufficient power to handle an average train under working conditions, and that the generating machinery was not powerful enough, it was considered advisable, in order to secure an accurate comparison with a steam locomotive, to detail one to pull the same train of cars over the same track, using hand brakes and reducing the running time to that of the electric motor. A number of tests were made on two occasions, some at slow speed, some on schedule time, and one run was made to ascertain the best performance of the engine, which, incidentally, propelled the train at an average net speed of 16⅔ miles an hour, making all the stops.

The locomotive, No. 67, was by no means the best on the line, and was not in the best condition, as it had a few days before been equipped with new brasses all around, and newly planed valve faces, all of which require plenty of time and oil to bring their surfaces to a good fit and polish.

The cylinders were 11-inch bore x 14-inch stroke. The four drivers were 36¾-inches diameter. The concentrated working weight on drivers was 23 900 pounds. The total working weight was 37 900 pounds. The maximum boiler pressure was 140 pounds per square inch.

Plate LVII records the results of a test made in slow time, June 20th, 1889, at 10.26 P. M.

The same corps of observers made the test, using the same instruments and methods. The writer used a new 60-pound spring in the Tabor indicator, which was driven by an arm and cord and thrown in or out of gear by the pawl and ratchet attachment used for high-speed work. During the time that steam was on, cards were taken on the average every nineteen seconds.

The average indicated horse-power (Diagram 1) was 25.92; the average net horse-power was 21.95 (Diagram 2), making the efficiency of the prime mover .85.

The net speed and net horse-power are both lower than in the electric motor test, as the speed was three-tenths of a mile per hour less. In this latter test the train had made several trips before this particular test was made, and the journals were consequently in better condition than they were in the electric motor test shown on Plate LVI, when the train

had stood idle on the track some days before making the test run, and had only made one preliminary trip on that evening.

In order to arrive at a conclusion respecting the relative economy of the two systems of motive power, the writer assumes the most tangible measure to be the cost of producing one net horse-power at the draw bar. Under actual working conditions, the locomotives develop on the average one indicated horse-power per hour with 6 pounds of good broken anthracite coal, which costs about $4.20 per long ton on board the engines of this line. In a short time this expense will be largely reduced by the new coaling plant at the Harlem River. This coal is very good, averages a pound per lump, and generates 7 pounds of steam* per pound of coal, in the average locomotive boiler. The city charge for Croton water is $1 per 1 000 cubic feet.

The machinery used in the electric plant was of an economical type, but could have been better still by investing more money in the plant. Assuming that the engine was compounded and used high pressure steam, and assuming that the plant could have been located on the water front so as to obtain condensing water, and assuming that an inferior coal was burned, a high average figure would be the development of one indicated horse-power per hour with 3 pounds of coal. Assuming that the inferior small coal would cost, after unloading, trimming, storing, rehandling and delivering at furnaces, $3 30 per long ton, and assuming an hourly consumption of steam of 20 pounds per horse-power, with an efficiency of .15 from the prime mover, the cost of 1 net horse-power by electricity would be :

Let $x = \frac{\text{I. H. P.}}{\text{N. H. P.}}$ = indicated horse-power required to produce one net horse-power at the draw-bar of motor ;

" a = pounds of coal burned per indicated horse-power hour ;

" b = pounds of water evaporated per pound of coal burned ;

" c = total cost of coal per pound at the furnace ;

" d = cost of water per pound :

then $x\ a\ c$ = cost of coal, and $x\ a\ b\ d$ = cost of water, which are respectively $ .03 and $ .001, making the total cost per net horse-power hour $3\frac{1}{10}$ cents. In the case of the locomotive ($x\ a\ c$ = $ .012) + ($x\ a\ b\ d$ = $ .001), = total cost per net horse-power hour, $1\frac{3}{10}$ cents.

The ratio of cost by locomotive to electric motor is 1 : $2\frac{38}{100}$.

* Total evaporation.

This is entirely exclusive of extra force at generating stations and additional repairs, and there could be no reduction in the working force on board the motors, whether steam or electric.

The foregoing estimate can only hold good with reference to the electric motor at a speed of $9\frac{4}{10}$ miles per hour, for beyond that the ratio of indicated to net horse-power rapidly increases, as shown by the diagram on Plate LX. Carrying out the curve of apparent increase of indicated horse-power per ton of train from 12 to 15 miles per hour, it seems very fair to assume an expenditure of 9 indicated horse-power per ton of train at that speed, which would bring the ratio of cost up to 1:4.7—by applying the previous method.

The question may be asked, what would have been the efficiency, if, instead of one train being run on the trial section, a number of trains had been run? The line resistance and other demands for power when no train was being moved would probably have remained the same, namely, 38.2 horse-power; this constant loss would have been divided among, at most, four trains on this section, which would reduce the average horse-power per train, at the low speed of 9.4 miles per hour, to 166.5 horse-power, while the net horse-power required would remain 26.7, giving an efficiency of $22\frac{9}{10}$ per cent., but, on the other hand, the evidence goes to show that if the speed was brought up to the regular average of 15 net miles per hour, the efficiency would have shrunk to about 8 or 9 per cent., and this would again be reduced by lowering the potential of the current to a safe limit.

A conservative estimate, based on these facts, would be that the cost of direct electric propulsion would be four times that of steam locomotion.

DISCUSSION.

A. M. Wellington, M. Am. Soc. C. E.—The motor tested was a Daft, was it not?

Mr. Moss.—Yes.

Mr. Wellington.—It seems to me, in a general way, that these diagrams, while no doubt entirely correct, would be liable to give a false idea of the relative promise of electricity and steam as a motor. One has only to compare the two rows of diagrams to see where the great loss in the use of electricity came in; it was in the conversion of the current of electricity into power on the train. I presume there was nothing special about that dynamo?

Mr. Moss.—The motor dynamo, 128 horse-power, was specially built for this motor, replacing the former 75 horse-power motor; the four generators, 200 horse-power, were standard electric light machines.

Mr. Wellington.—In a general way it may be said that there is now readily realized from 80 to 90 per cent. of the power put into the steam engine by good dynamos. The electric current can be and is, under favorable conditions, converted into power with a loss say, of not much over 10 per cent. So under favorable conditions, to bring out the power consumed, 60 or 70 per cent. of that developed by the engine can be given out by the motor. This motor was evidently defective. Mr. Daft is entering into a new and unknown field, where certain mistakes and failures are inevitable, but we can see it would be an entirely different thing if, on the diagrams, that upper one were reduced so that it was only in the ratio of 100 to 60 or 70 to the power below.

I don't know how soon such economy is to be realized in railway motors. We evidently are not near it yet, but it can be counted on that even now appliances exist which can make a much better showing than those tested in these experiments.

Mr. Moss.—The system under consideration embodied all the experience of the original experiments made several years prior to the tests described, the motor in particular having been entirely rebuilt. Originally it had 75 horse-power, and drove one pair of drivers by friction wheels, but was changed by putting in a specially designed motor of 128 horse-power, which drove four coupled wheels by steel gears and phosphor-bronze pinions. The methods of regulation were similar to those used in the best street railway practice. External resistances, while necessary at times, were used as little as possible. Much of the regulation was automatic, as shown by the diagrams, when the motor was climbing up-grade. As the train resistance increased and the speed slackened, the counter current decreased, and automatically permitted more current to flow through the motor, increasing its pulling power and decreasing the efficiency. Another method of regulation was to vary the strength of the field, by many considered to be preferable to adding external resistance. Two other electric systems were tried previously on the elevated roads—one using a differential motor, the other applying power to the car axles, but to Mr. Daft and his very able lieutenants belongs the credit of advancing further along the road to a practical solution of the problem of electric elevated railways than any other experimenters have.

Alfred E. Hunt, M. Am. Soc. C. E.—My experience coincides with the statements of the last speaker, and not with those of Mr. Wellington. I agree with Mr. Wellington, however, that the best form of generators or dynamos are capable of showing an efficiency of nearly 90 per cent., measuring the force by the formula given in the paper, namely,

E. H. P. $= \frac{\text{volts} \times \text{amperes}}{746}$, that is, that the most improved forms of apparatus for converting energy in the form of mechanical power into energy in the form of electrical currents, have succeeded in reducing the loss in conversion to about 10 per cent. But the tests of efficiency of generators, or of machines for converting energy in the form of electrical currents into energy in the form of mechanical power, under various loads and different speeds, have not given as satisfactory results under practical working conditions, whether the tests are made by friction-brakes, transmission dynamometers, or by making the motor drive a dynamo which supplies it with current, and measuring with a dynamometer the mechanical work of the auxiliary engine, or the other power needed in addition to the motor, in order to give the dynamo the capacity for the required electric current. I am aware that many manufacturers claim an equal efficiency for their motors with the results of the best generators—an efficiency of some 90 per cent. However, it has not been my experience that they are warranted in this claim, if the tests are made with the conditions of actual working loads, such as the motors will receive in actual service; especially has this been found the case if the tests include frequent stopping and starting of the machines such as they very often undergo, and particularly in such cases as cited in the paper.

When a motor is running, part of the electric energy of the supplying current is spent in doing work, and the remainder is wasted in heating the wires of the circuit, and as internal resistance of a dynamo machine, when warmed through with continual working, is considerably higher than when it is cold, and, further, as the efficiency of a motor depends largely on the relation between the electromotive-force which it itself generates while rotating, and the electromotive-force of the current which is supplied to it, it seems practically impossible to obtain, under circumstances as given in the paper under consideration, nearly as efficient service from motors as from generators working under constant loads.

Mr. Wellington.—I think Captain Hunt misunderstands the statement. I did not say that it was realized in any motors of this kind; I said it "had been realized." I have seen within a week the record of a test of an electric motor which did develop something over 80 per cent. of the power put into it by the electric current, and still more has been obtained in other tests.

Referring now to another point: Mr. Moss speaks of the difficulty in starting; that is undoubtedly one of the great difficulties with the Daft plants and many others, but that difficulty is not likely to be permanently unavoidable. There is already one plant that I know of which illustrates what can be done; in that plant the motor does not stop at all when the train stops, so that the waste from this cause is much less. This motor

obtains the extra starting power by an exceedingly ingenious method of varying the speed. It is too complicated to attempt to describe here offhand; in a general way, the result which is accomplished is that there are two wheels adjacent to each other, grasped by brakes. In starting the brake is thrown into one position, a fixed one, and one of the wheels is held stationary. This gives a slow starting speed. After starting the brake is simply reversed; the wheel that was stationary is allowed to move and the other held stationary. The speed is then increased and power decreased four or five times. This device promises much advantage when finally perfected, yet it merely illustrates what may be done by future improvements.

Oberlin Smith, M. Am. Soc. C. E.—I do not think that we ought to be discouraged as to the future of electric locomotion by these figures, although they explain very well, perhaps, why the New York Elevated Roads have not gone on and applied electricity upon their lines. It seems to me that a part of the trouble is in trying to run with an electric locomotive at all. From what Mr. Moss said about the burning of the armatures, it looks as if some of the diffiulty was there; that the locomotive was not at all proportioned to the weight of the train. If some proper system of applying the motors to the cars themselves had been used, it is probable that very much better results would have been obtained; and I decidedly think that in the future, when we do run cars or trains by electricity, we must do it in that way. The total dead-weight can then be proportioned to the number of cars used at any given time, and although we should build our cars much lighter, we should get abundant traction—and without carrying several tons of ballast in the shape of the unnecessary monster in question.

We are not told anything about the particular "system" used in these experiments, but it must have been very imperfect, in comparison say, with our best electric street railways, to show so low an efficiency. It seems as if with a good dynamo—one having 85 or 90 per cent. efficiency, and a motor as good—there should have been far less loss. There is the loss, of course, in the line, and perhaps the greatest part of loss may be in the line, but my impression has always been that we may expect about 50 per cent. of our power at the car axles. If we got only 40 per cent. in this case of the energy put into the dynamos, we would see a very great change in the diagrams.

If we look at the principle of the thing we have, besides the advantage of generating our power so much cheaper "on shore," the chance (if we choose) of dispensing entirely with locomotives and with the most of their dead-weight. I think there ought to be a great deal of experimenting done in this line, and that it ought to be tried on the elevated roads, as a feeler toward the use of electricity upon our ordinary railways. The elevated roads in a city are peculiarly well adapted to this kind of traction, certainly more than the surface street roads can be; and yet the latter are a magnificent success.

James B. Francis, Past President Am. Soc. C. E.—Sir William Thomson stated to a committee of the English Parliament in 1879: "I look forward to the Falls of Niagara being extensively used for the production of light and mechanical power over a large part of North America," and in a deferred answer to questions regarding the quantity of metal required for the electric transmission of energy, he stated, as the result of his calculations, "a copper wire of half an inch diameter suffices to transmit 21 000 horse-power to a distance of 300 statute miles, with an expenditure of 26 250 horse power at the sending end. The 20 per cent. loss, being 5 250 horse-power, is spent in generating heat all along the 300 miles of conductor, which is at the rate of 17.5 horse-power per mile, or 1 horse-power for 300 feet."

The weight of a statute mile of round copper wire of ½ an inch in diameter is 4 000 pounds, of which the cost is about £200. Hence the cost of the copper to transmit 80 per cent. of 26 250 horse-power to a distance of 300 statute miles (say from Niagara to Montreal, or Boston, or New York, or Philadelphia) is £60 000, or less than £3 per horse-power actually transmitted.

I have made many inquiries as to the extent to which these anticipations have been realized, practically, and find it to be, so far, very small.

APPENDIX.

In order to compare the relative cost of equipment and operation of this line, by steam direct or by electric current, the following estimates of cost and operation of motive power are appended, and a sketch of the proposed generating plant (Plate LXI) on the direct current system, with the motors in parallel, voltage 400 to 500. Although the electric plant used in the tests had been enlarged and improved during the years of experimenting on the road, and was considered to be nearly perfect, it is conceded in the following estimates that it would be possible, by the greatest refinements known, to do the work with but *one-half* the expenditure of energy found in the tests, which were the latest and most favorable and taken in dry weather. Much depends on the accuracy of the diagram showing the increase of power expended to move 1 ton on the track at different speeds; if the generators or motor were overloaded it would be unfair to assume the line of increase as being necessarily correct for future practice.

To avoid constructing an overloaded diagram, the right-hand ordinates were calculated from runs with but two empty cars, while the left-hand ordinates were calculated from runs made with four empty cars. The motor had 128 horse-power, according to the builder's Prony brake tests, which exceeded the maximum power of any locomotive on this line, and the generators had collectively 200 horse-power backed up by an engine of 400 horse-power—maximum power; as the net horse-power required in the most extreme case on the diagram was but $16\frac{32}{100}$ horse-power, the great surplus of power is self-evident. In this case the engine exerted an average of 187 I. H. P., losing 10 per cent. in itself and the shafting; it delivered 168.3 horse-power to the dynamos, which, with an efficiency of .85, delivered 143.1 horse-power to the line; the net line loss was 29.1 horse-power, which leaves 114 horse-power delivered to the motor, or more than seven times the net horse-power required to propel the two empty cars, so that there can be no question as to the truth of the curve of increase. The Ninth Avenue section of the Manhattan Railway is 5.06 miles long, which, with double track, sidings, cross-overs and turn-outs, aggregates 11.45 miles of single track.

To save first cost and operating expenses, the power is concentrated at a point in the middle of the line, near the Hudson River front, so as to obtain free condensing water and to handle coal throughout by the most improved machinery. The buildings would be plain brick, and the construction throughout would be as cheap as consistent with thorough construction and the highest efficiency.

Coal would be unloaded by automatic steam shovels, conveyed to the power house, elevated and distributed by steam power, into bunkers over the boilers, saving a coal yard, and permitting the coal to trim itself by gravity and fill the automatic stokers as wanted, by opening

gates in the down spouts. The ashes would drop by gravity into conveyors, to be delivered automatically on the ash heap. This construction makes it possible to use a low grade of anthracite with 33 per cent. of soft coal, which is necessary to light up the fine hard coal in mechanical stoking. The mixture would cost $2.25 per ton at the furnace mouths. The boilers would be sectional tubular and worked at 180 pounds per square inch. The evaporation with this inferior coal would be from 5 to 7 pounds of steam (perfectly dry) per pound of fuel. The engines would be vertical to save room, triple expansion and surface condensing, of the Corliss type, running at 100 revolutions per minute, the most economical type known. The surface condensers would nearly wipe out the Croton water expenses. Such engines would consume 13 pounds of steam per horse-power hour, but on such variable and unsteady loads would average about 16 pounds per horse-power hour.

The shafting would be arranged in sections and the pulleys would be connected by friction clutches.

The dynamos would be located above the engine room so as to save ground space and pull against the main pulleys, of various types, and fully equipped with switches, lamps, resistances, ammeters and voltmeters, cut-outs, lightning arresters, etc., and all operated from one switch-board.

It would be necessary to carry the current to and from the structure in a subway. The general arrangement of conductors would be the same as in the tests, but the current would be returned through both lines of rails instead of one line.

The motors would certainly require to have 250 horse-power, and probably would be driven by gears, which still retain the supremacy in electric railway motors. The motors would have four coupled drivers and a truck, in order to distribute the 20 tons of weight over the light structure of this line. The present thirty-five locomotives, 16 to 19 tons each, are too light for service on the other lines, and would be sold. They bring when sold one-third of their original value. The present standard engine has about 240 horse-power and weighs 47 000 pounds.

As the cars are now heated by live steam from the locomotives, it would be necessary to re-equip them with, preferably, low-pressure steam heaters, so as to utilize the present radiators, pipes and hose connections. They would require good anthracite coal, such as is used to heat the stations.

The present steam vacuum brakes could be utilized by adding a vacuum tank to each car, and placing vacuum pumps on the car or motor axles or driving them by electric motors.

It would not be advisable to substitute incandescent lights for the present oil lamps, on the grounds of additional outlay and cost of operation, and the transfer of cars to and from other divisions not equipped with electric current.

The shop room now occupied for locomotive and car repairs would be needed for the same number of cars and motors, and some new tools would be required for making and repairing electrical machinery, gear cutting and line work.

As the repair shop has no spare room and is 7 miles from the center of this section, it would be necessary to build a small repair shop alongside the power house (not shown on sketch), containing tools suitable for repairing boilers and steam pipes, boring and facing cylinders, shaft and pulley turning, shaft seating and pulley boring, also machinery for repairing dynamos, etc. A second floor would be added in which would be the switch-board, engineers' and clerks' offices, storerooms, etc.

The present shop expenses (aside from locomotive repairs) would be common to both systems, and are not included in either estimate.

The maximum power required is in the afternoon service, when twenty-two trains are on the road at once. The train schedule shows fourteen trains starting at one time, three running with power between stations and the balance standing or moving and using no power.

The power curve shows that four times as much power would be required to operate the trains at 15 miles per hour as at $9\frac{4}{10}$ miles per hour; but assuming that this could be reduced one-half, and deducting all line losses, the power required would be 7 416 horse-power; adding to this 4 horse-power per train for line losses, engine, shaft and dynamo losses; 315 horse-power for braking, and 980 horse-power for operating electric lights at station, steam shovel, conveyors, elevators, air, feed and circulating pumps, steam stokers and shop engine and tools, we have a total of 8 779 maximum horse-power required.

The present consumption of coal is 61.3 tons per day, which includes 4.7 tons consumed in steam-vacuum braking, making 6 000 stops daily, and an average daily charge of 15.62 tons for car heating, which leaves 41 tons of coal daily chargeable to propulsion of trains. With an average production of one I. H. P. per hour with 6 pounds of coal, the average daily aggregate horse-power for propelling and braking trains by locomotives is 16 787, of which 15 per cent. is lost within the locomotives, leaving the net aggregate horse-power 14 269 required to operate the road daily, heating excepted. To obtain this power by electricity with a consumption of $2\frac{1}{4}$ pounds of coal per I. H. P. hour, and charging 7.09 I. H. P. per net horse-power recovered on the track, as shown by the tests, would require 101168 I. H. P. at the generating station, which would require $101\frac{2}{3}$ tons of coal daily. Car heating will average 15.6 tons of station coal daily. Great care and judgment were necessary in handling the motor, and two men were required to operate its mechanism. Hence the writer considers the present engineer and fireman would make a competent crew. It is imperative that two men should be aboard of each motor, even if all the regulating and operating mechanism could be handled by one man.

Construction Charges.

Real estate, twenty-five lots, at $15 000	$375 000
Buildings be acquired and torn down	40 000
Excavating old walls, earth and mud, including pumping, at $1 per yard	28 000
Sheathing and timbering excavations	2 000
Piling, driven, cut and capped, at $10 in place	42 000
Concrete walls, piers and foundations, at $6 per yard,	75 000
Back-filling, curbing, paving, and flagging	2 000
Portland cement floors, at $20 per yard	35 000
Brick fronts, walls and arches, at $20 per thousand	120 000
Stone sills, caps and coping	2 000
Two chimneys, 110 feet high, including foundations, at $40 per thousand	20 000
Columns and brackets	5 000
Rolled floor beams in place and bolted, at 5 cents per pound	44 650
Roof trusses and purlins, at 8 cents per pound, in place.	50 800
Corrugated galvanized iron roofing, No. 18, at 15 cents per square foot	10 100
Skylights, gutters and leaders	2 000
Coal-pocket beams and ¼-inch steel lining, 5 cents per pound	27 240
Brick chimney connecting flues, at $25 per thousand,	7 620
Brick setting for boilers, at $25 per thousand	18 950
Sashes, doors, closets, offices and other carpenter work.	5 000
Plumbing	2 000
Painting	3 000
Croton water connections, tanks, meters, etc	4 000
River connections for condensing water, including trench and pipes to and from surface condensers,	15 000
Brick conduit from power house to railway to carry + and — conductors	31 000
Sectional boilers, twenty of 600 horse-power each, at $12 000, including reserve	240 000
Feed pumps and injectors	5 000
Boiler fittings, including damper regulators	7 500
Steam piping to engines, including copper bends and corrugated expansion joints	24 000
Seventeen triple-expansion, surface-condensing, vertical Corliss engines, 600 horse-power each (with 40 000 pounds 16-feet fly-wheels, 100 revolutions per minute, including jackets, air pumps, gauges and fittings, and reserve engines)	289 000
Exhaust piping to condensers, copper	6 000
Seventeen surface condensers, at $2 500	42 500
Circulating pumps	13 000
Feed heaters (live steam in conjunction with hot well overflow)	9 000
Shafting, hangers, pulleys and friction-clutches	48 000
Belting	15 000
Dynamos, 11 000 horse-power, including reserve power; wiring, switches, lamps, resistances, lightning arresters, voltmeters, ammeters, cut-outs and switch-board	455 000

Copper conductors, 68 000 lineal feet, 4.66 square inches section, machined, jointed and laid, at 25 cents per pound	$305 250
Seven thousand insulated clamps for supporting conductors	12 000
Cross-wiring and connecting rail ends for conductors,	3 500
Thirty-five motors, 250 horse-power each, at $18 000 complete	630 000
Additional tools required at repair shops, for electrical work, gear cutting, shaft and pulley turning, cylinder and pulley boring, and including shafting, belting in small repair shop adjoining power house	40 000
Car heaters to replace present steam heating plant in cars	92 500
Power brakes for cars and motors to replace steam vacuum brakes	42 500
Lighting plant for power house and repair shop	5 000
Loss on sale of thirty-five locomotives too light for service on other branches of the Manhattan system (will bring one-third of original value, $4 000)	93 350
Labor setting up engines, boilers, pumps, piping, condensers, shafting, pulleys, clutches, belting, dynamos, coal shovels, conveyors, stokers, etc	20 000
Freights, lighterage and transportation of materials	15 000
Automatic stokers, including down-spouts, branches and gates, engines and shafting, hoppers, grates and ash-chutes	15 000
Automatic steam shovels, conveyors, elevators, distributors, ash conveyers and driving engines	61 000
Steam pipe covering	4 000
Two electric overhead cranes in dynamo room, capacity 15 tons, with tracks and girders, four traveling electric hoists in engine room, and two traveling overhead electric cranes in repair shop.	8 000
Total	$3 468 460
Contractors' profits and contingencies	346 846
Engineering and superintendence	40 000
Total construction charge	$3 855 306

Estimate of Operating Charges Against Motive Power.—One Year, by Steam Locomotives.

Yearly interest, at 6 per cent., on locomotives, coal and water stations	$9 000 00
Interest on present shop and tools not included in either estimate, and interest on present heaters and vacuum brakes is not included in either estimate.	
Depreciation and repairs	24 472 97
Total cost of coal on board locomotives, consumed in propelling trains, heating and braking	90 914 77

Oil and waste	$2 783 84
Croton water tax and expenses	4 701 41
Dispatchers and clerks	537 42
Engineers and firemen	84 213 49
Other labor, hostlers, wipers, etc	9 725 90
Water and coaling stations	711 30
Operating charge, motive power, one year	$227 061 10

OPERATING CHARGES AGAINST MOTIVE POWER—ONE YEAR, BY ELECTRICITY.

Yearly interest on construction outlay	$231 318 36
Depreciation and repairs of plant, average at 10 per cent	385 530 60
Car heater coal at $4 per ton on car	22 800 00
Coal burned under boilers for propelling trains, for braking, power-house lighting, coal and ash handling machinery, shop machinery, and driving feed, circulating and air pumps: Total coal burned under boilers, 131 tons daily, at $2 25	107 583 75
Motor engineers, same as locomotive engineers	54 165 70
Motor assistant engineers, same as locomotive firemen	30 047 79
Additional force employed on repairs to generating plant, and additional force required to superintend and operate generating station, etc	60 000 00
Dispatchers and clerks	537 42
Oil and waste	5 000 00
Dock rental for coal barges	6 000 00
Removal of ashes	2 000 00
Croton water tax on water used in closets, washstands, cleaning and for making up losses in hot feed return from condensers	1 000 00
Motive power, operating charges, one year	$905 983 62

Thus, it seems that the cost of motive power by electricity would be about nineteen times as great as the cost of steam locomotives, and the cost of operation, even with the most advanced machinery, and with a concession of twice the efficiency found in the tests, would be 3.99+ times as much as with the present steam locomotives. The writer hopes to see great advancement along the line of progress in transmitting power by the electric current, and believes that a careful study of the weak points in electric transmission on the large scale of these experiments would result in many improvements. Enthusiasm will not solve the problem. All sound and careful engineers and scientists are anxious for the truth, and these tests and figures are offered as the result of an impartial carrying out of the writer's order to get "the truth, the whole truth, and nothing but the truth."

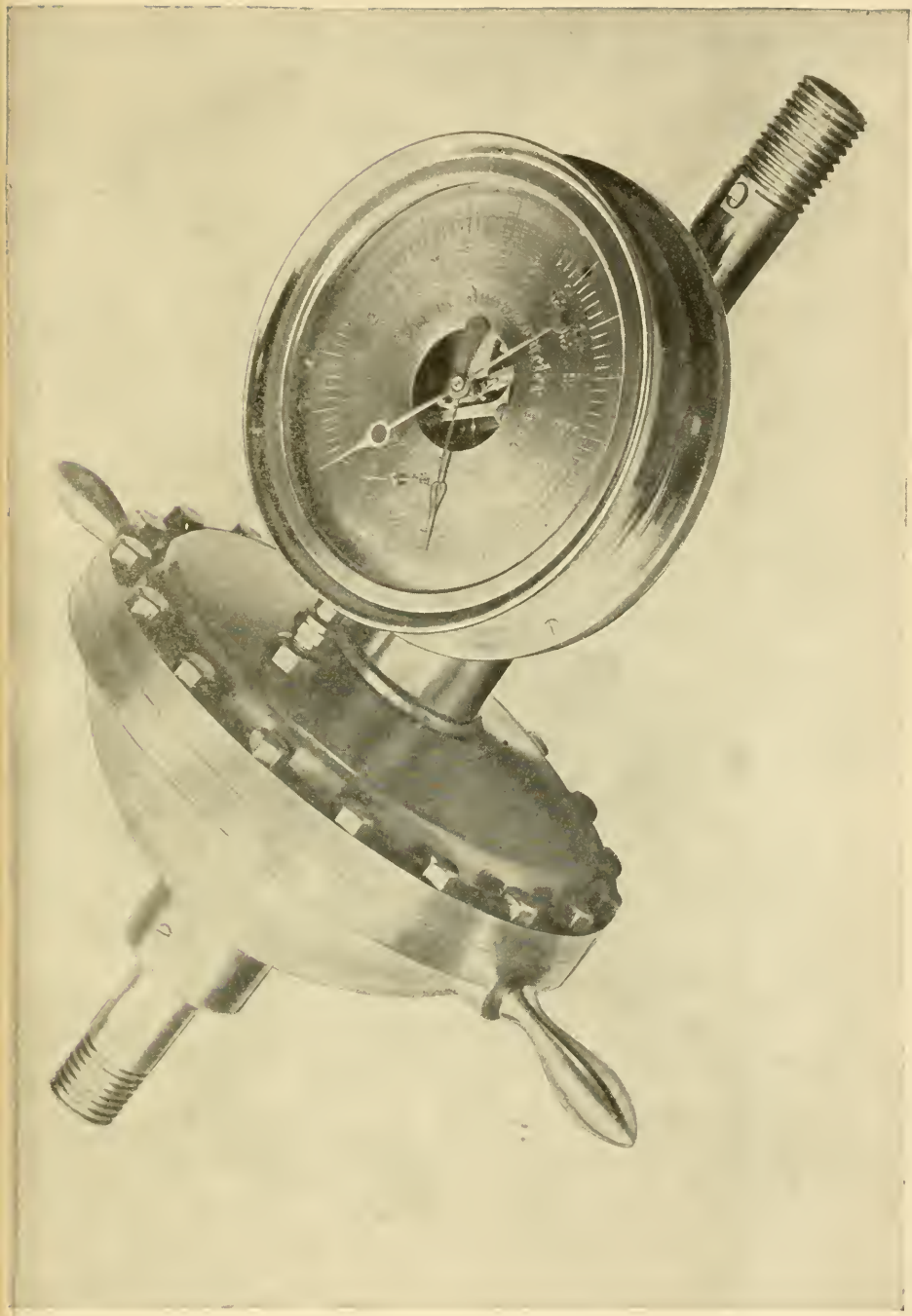
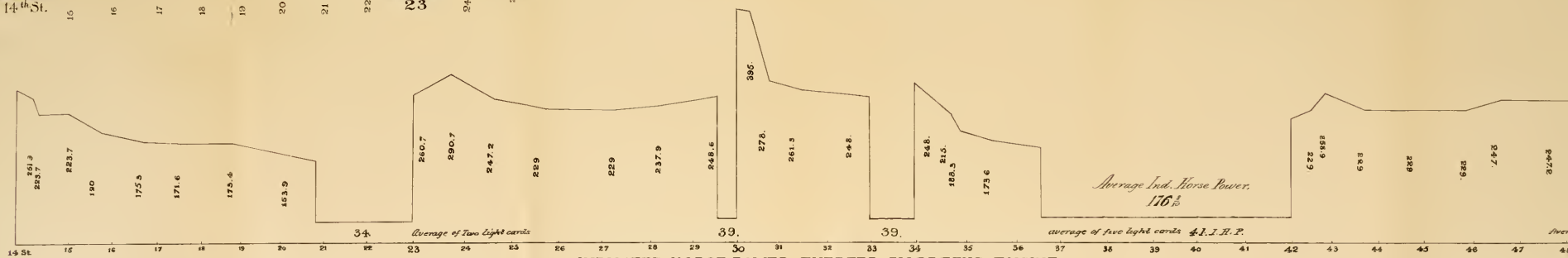
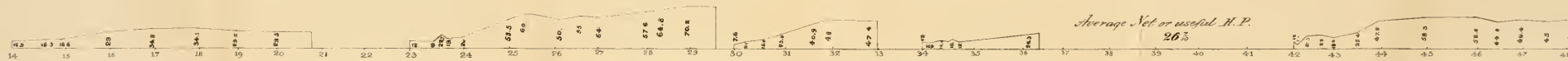


PLATE LV. HYDRAULIC DYNAMOMETER. Vol. XXIII, p. 216.

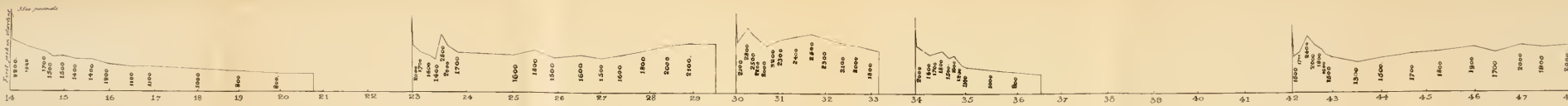
Test



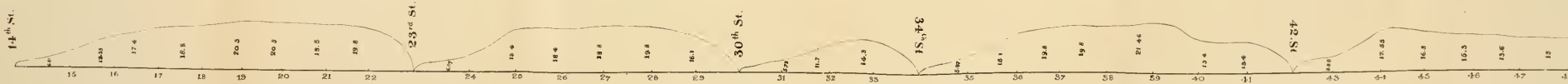
INDICATED HORSE POWER, EXERTED BY DRIVING ENGINE.



NET HORSE POWER EXPENDED TO PULL TRAIN.



PULL OF MOTOR, measured by DYNAMOMETER.

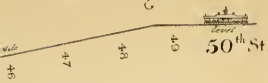


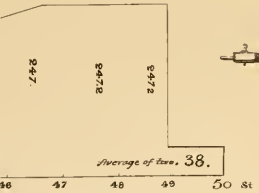
SPEED OF TRAIN, in FEET PER SECOND.

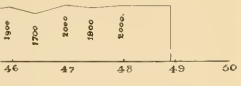
Test of Electric System.

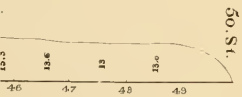
June 15th '89.

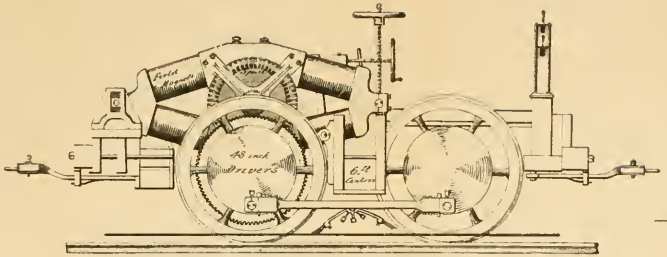
PLATE LVI
TRANS AM SOC CIV ENGRS
VOL XXIII No 453
MOSS ON
TESTS OF ELECTRIC MOTOR





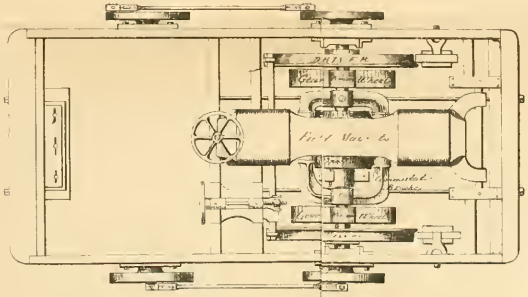




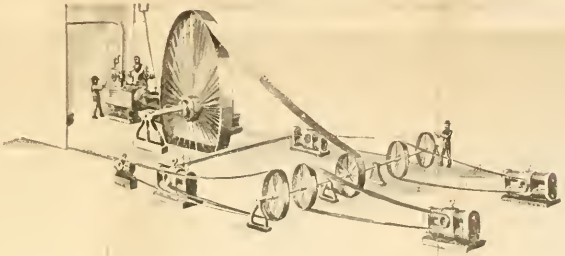


Driving Mechanism of Electric Motor
(Frame and Cab removed.)

EXPERIMENTAL TRAIN.
4 Cars, 29500 lbs each.
1 Motor, 20000 lbs.
Train Crew. 1800.
TOTAL. 69.9 TONS.

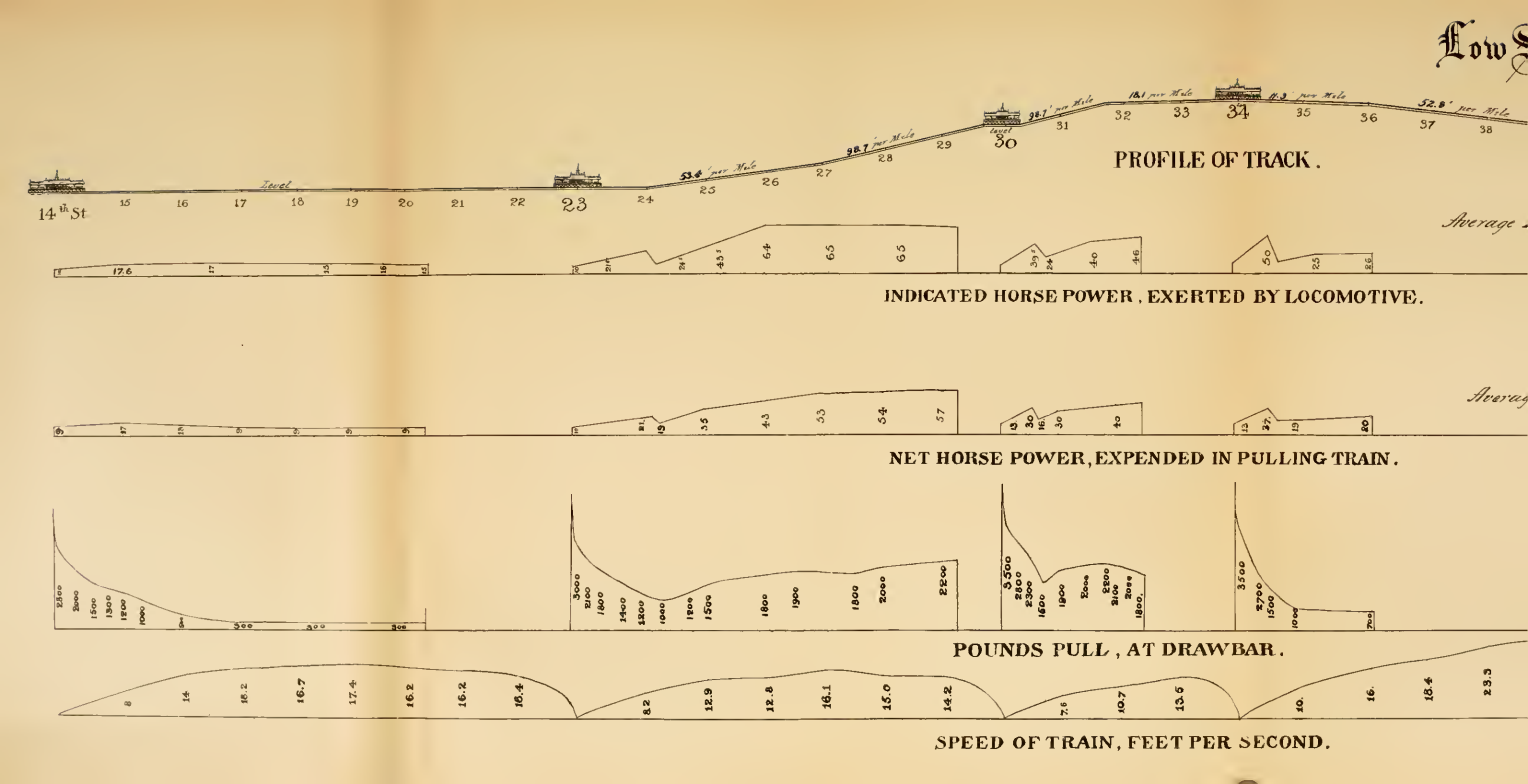


Plan of Electric Motor.

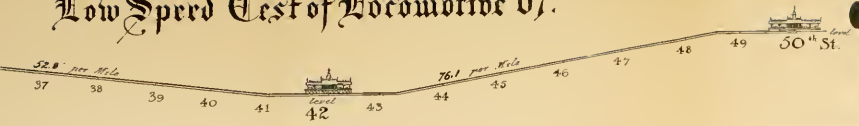


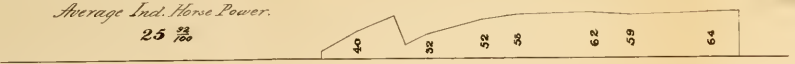
Sketch of Automatic Engine and Dynamos.

Engine Cylinder, 22" bore × 42" Stroke.
Revolutions, 90 to 100 per minute
Fly Wheel. 16 feet diam.

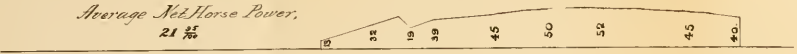

Low
PROFILE OF TRACK.
14th St
Level
53.4' per Mile
98.7' per Mile
98.7' per Mile
18.1 per Mile
11.3' per Mile
52.8' per Mile
Average
INDICATED HORSE POWER, EXERTED BY LOCOMOTIVE.
Averag
NET HORSE POWER, EXPENDED IN PULLING TRAIN.
POUNDS PULL, AT DRAWBAR.
SPEED OF TRAIN, FEET PER SECOND.

Low Speed Test of Locomotive 67.





VE.



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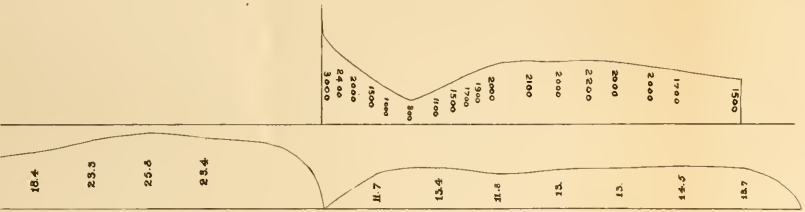

Motive Power used in ——— Electric System.

Three Boilers 100 Horse Power each - **300 H.P.**
One Engine, indicating **100** to **400** Horse Power.

Average Power available for propelling train
15 %

Maximum result, the 60 Ton train was propelled at an average speed of 9 1/10 Miles per hour. from 14th to 50th St. stopping at 23rd, 30th, 34th and 42nd Sts using hand brakes

The Locomotive used in the Competitive Trials.

Scale of each, 1/8" = 1 foot

PLATE LVII
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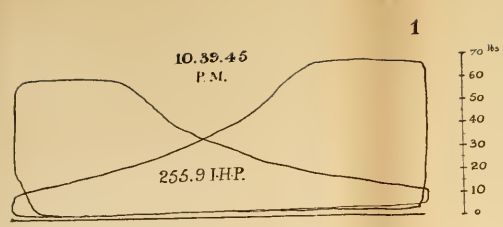
TESTS OF ELECTRIC MOTOR

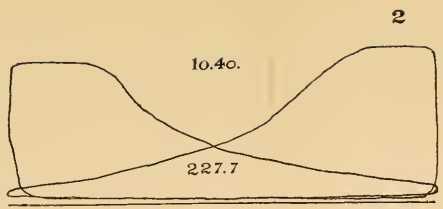
Indicated Horse Power, 10 to 65, to produce the same effect as the electric System.
Maximum Horse Power, 125.

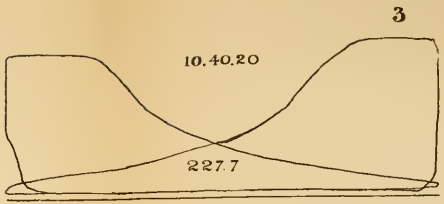
Average of Power available for pulling train.
85. per cent.

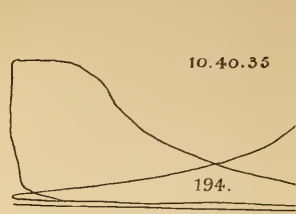
Maximum result, the same train, on the same track, making same stops.
by hand brakes, averaged 15. Miles per hour.
with vacuum brakes, " 17. " " "

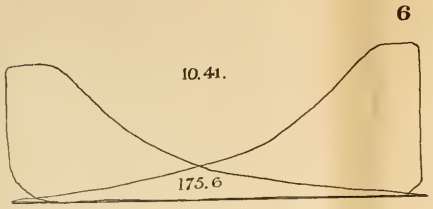
Indicator Cards taken from Driving Engine during Test of Electric M

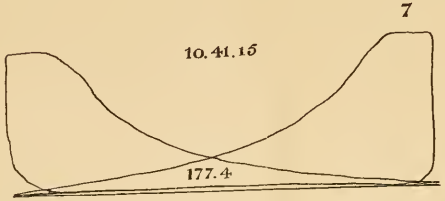


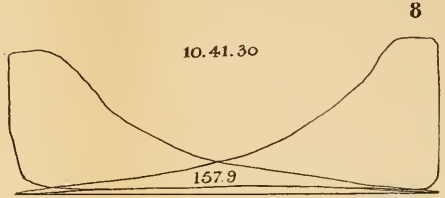


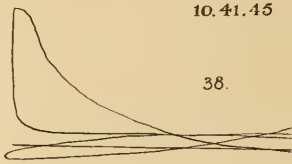


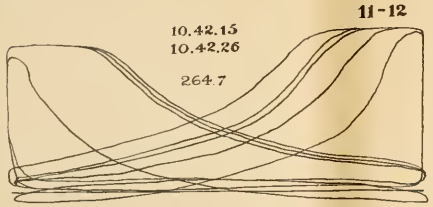


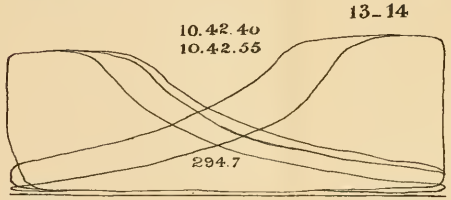


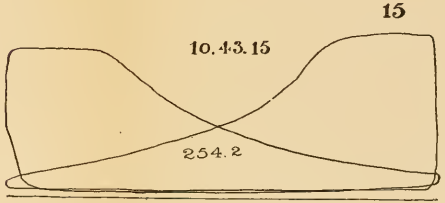


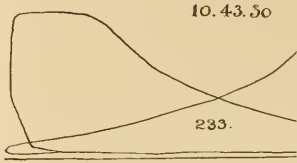




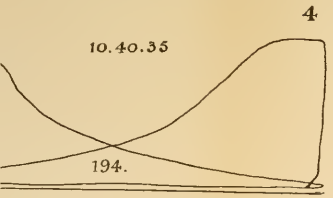


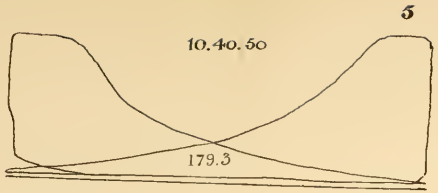


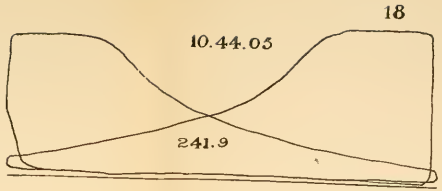


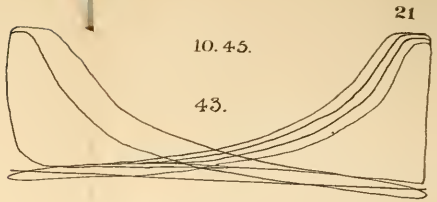


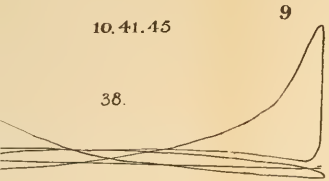
ing Test of Electric Motor shown on Plate LVI.

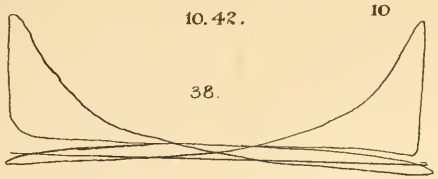


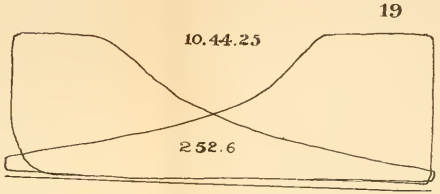


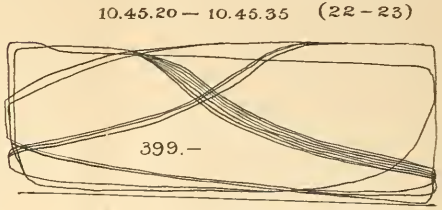


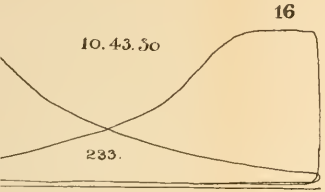


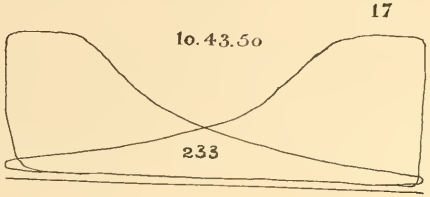


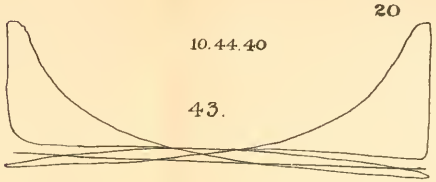


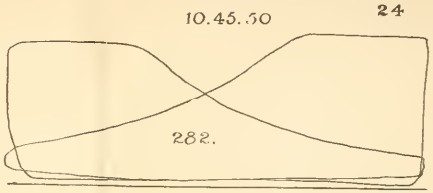




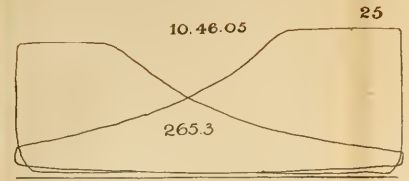


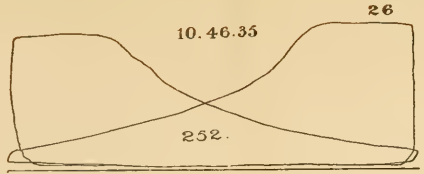


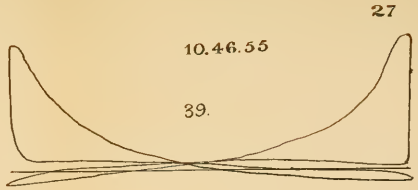


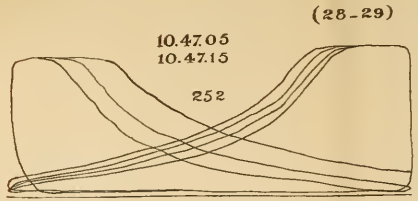


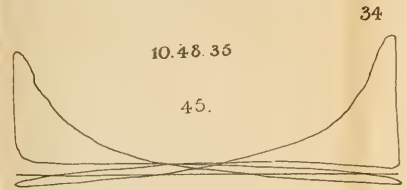
Indicator Cards taken from Driving Engine du

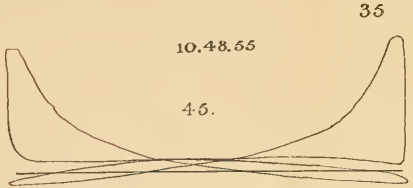


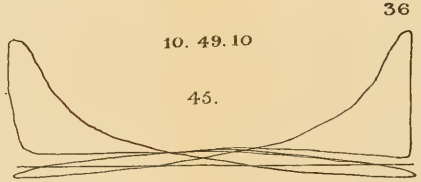


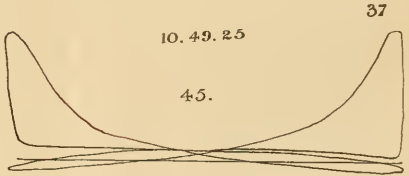


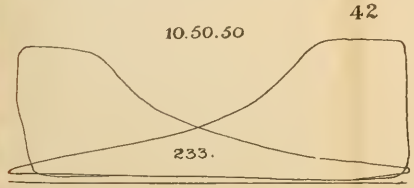


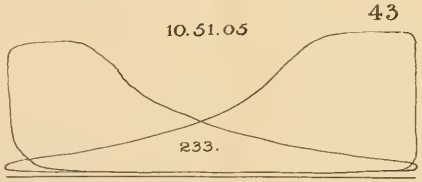


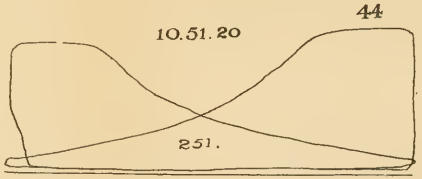


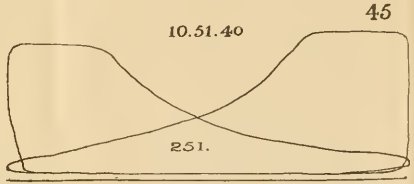












g Engine during Test of Electric Motor shown on Plate LVI

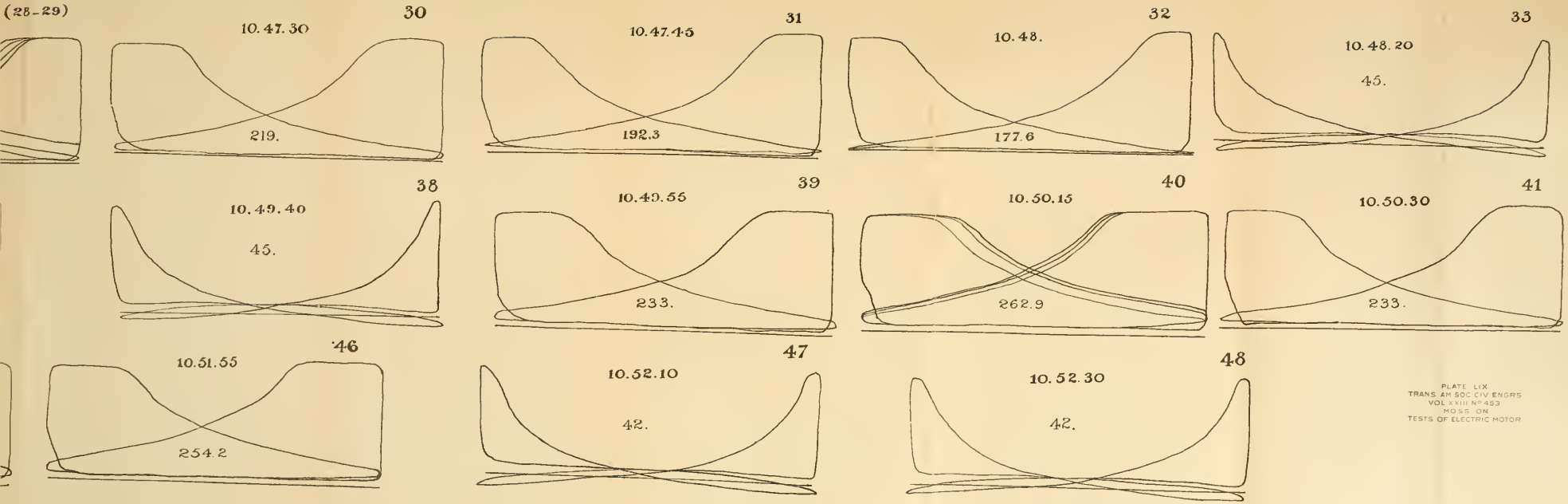


PLATE LIX
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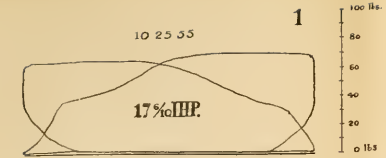
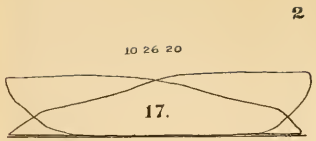
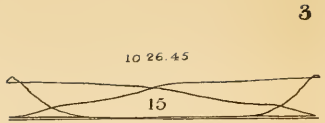
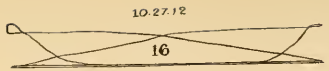
TESTS OF ELECTRIC MOTOR

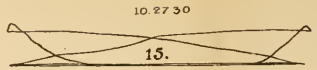
Indicator Cards taken during Low-speed Tr

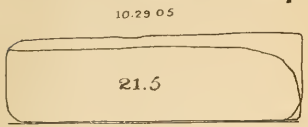


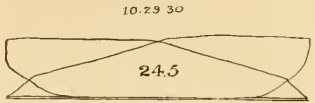


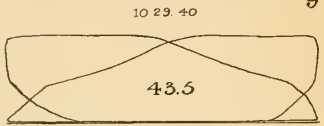


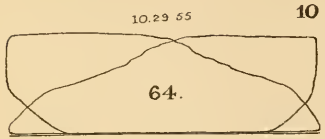


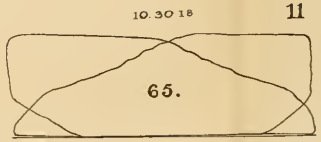


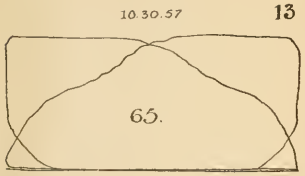


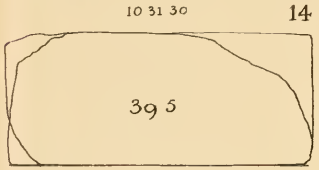


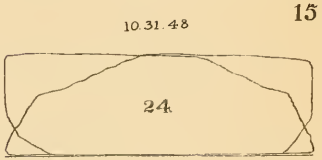


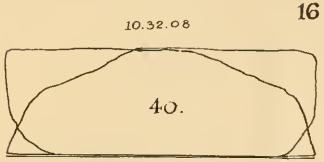


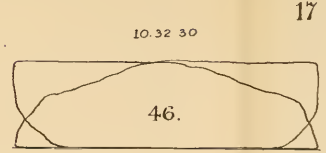


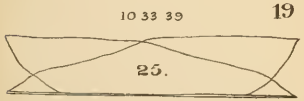


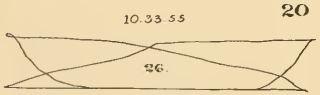


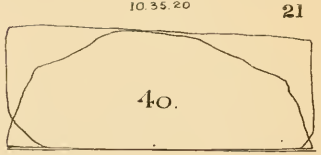


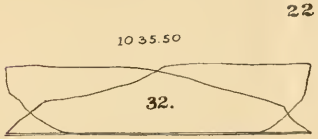


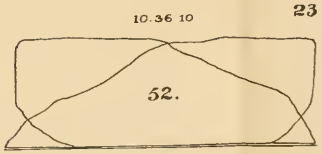












ow-speed Trial of Locomotive No. 67.

5

11

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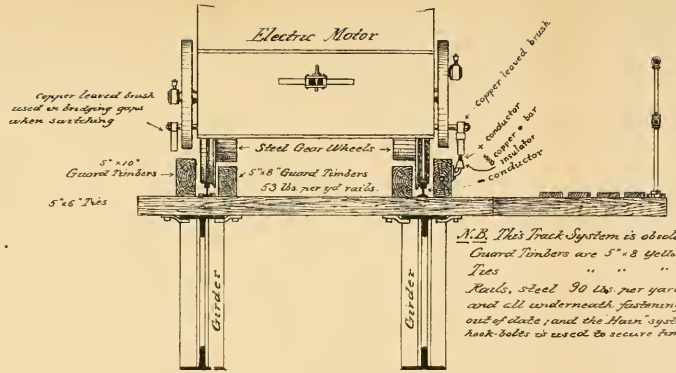
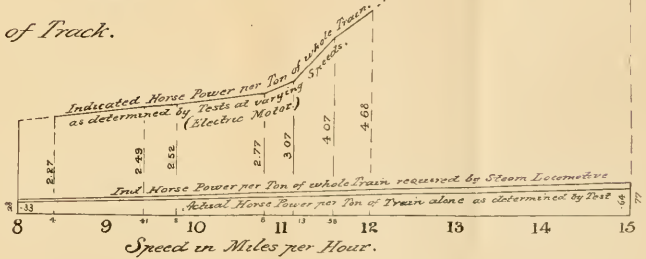


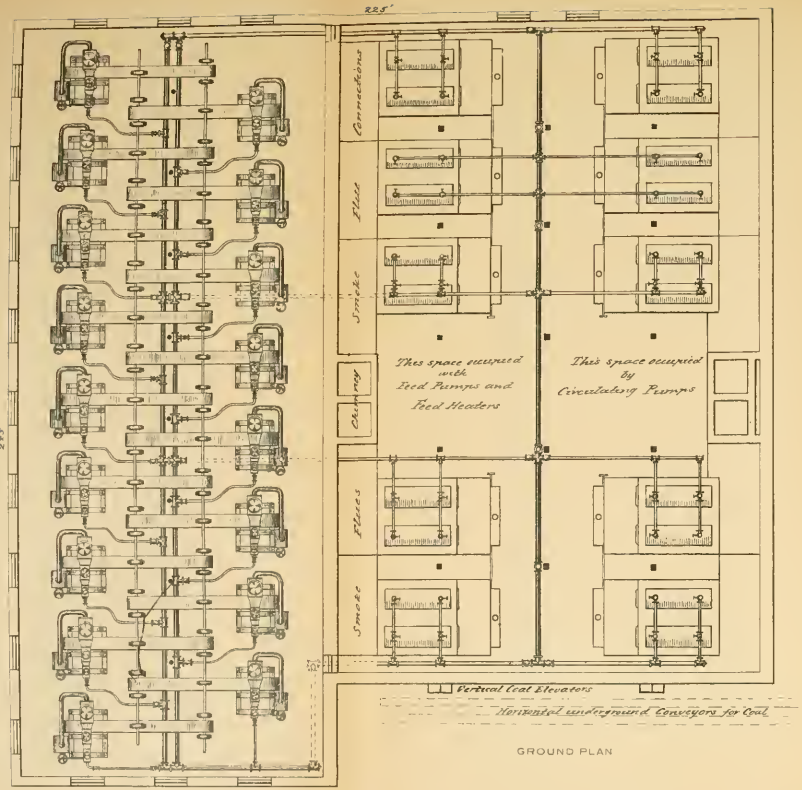
N.B. This Track System is obsolete.
Guard Timbers are 5"x8 Yellow Pine
Ties " " " "
Rails, steel 90 lbs. per yard
and all underneath fastening is out of date; and the "Hunt" system of hook-bolts is used to secure timbers.

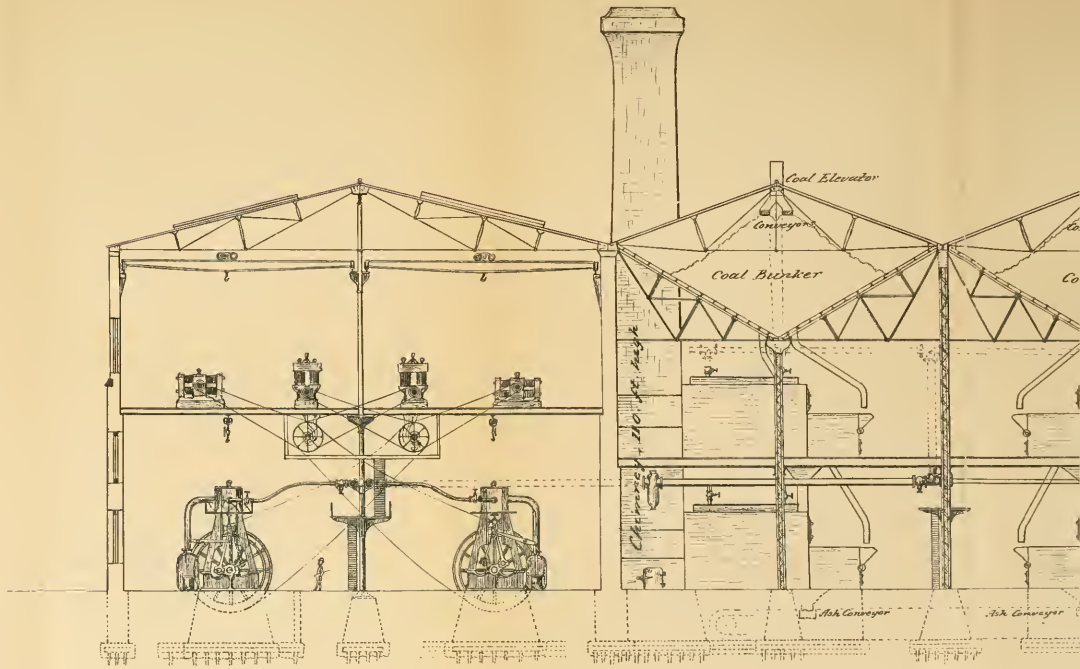
Section of Track.



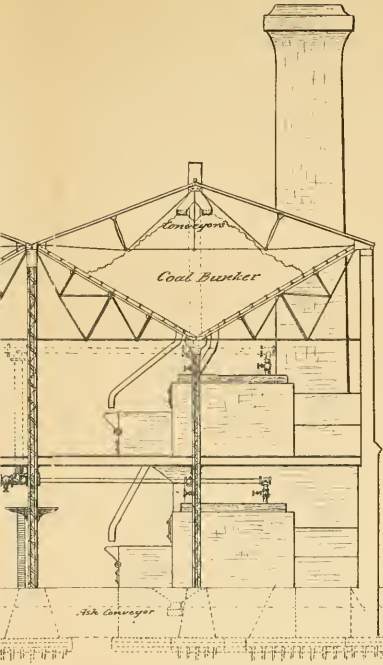
Power Diagram.

Sketch of Electric Generating Plant, sufficient for the 9th Ave.





he 9th Ave. Line.



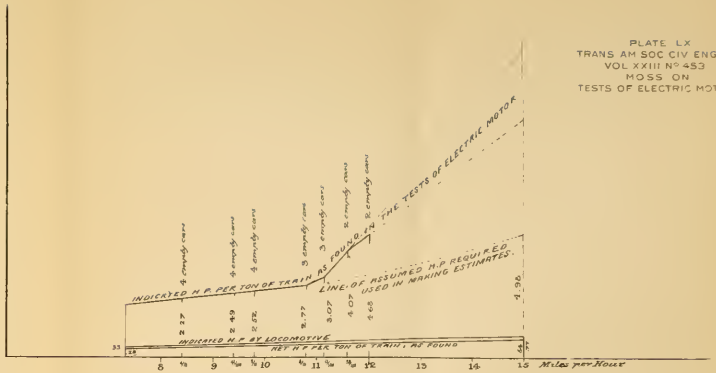
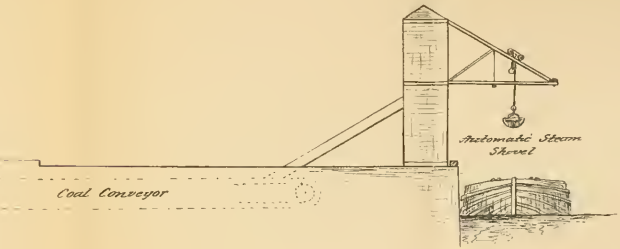


PLATE LX
TRANS AM SOC CIV ENGRS
VOL XXIII No 453
MOSS ON
TESTS OF ELECTRIC MOTOR



AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXIII.—November, 1890.)

IRRIGATION IN INDIA.

By HERBERT M. WILSON, M. Am. Soc. C. E.

WITH DISCUSSION.

In view of the interest which is at present taken in the subject of irrigation development in the West, it will be well to observe what other countries have done to increase the amount of their productive agricultural land by its means. While in this paper it is proposed to discuss the methods and results of scientific irrigation practiced in India, reference will be made only to such details as may furnish us useful examples because of the similarity of the conditions under which they exist to those to be found in our own country.

The necessity of a more thorough and systematic development of our irrigation resources has been already so frequently discussed that it need not be farther dwelt upon here, as it is generally admitted that the utilization of the waters of the arid West for purposes of irrigation on a concerted scientific plan will render productive many millions of acres which are now only barren desert, and will furnish homes and support for millions of people.

The conditions under which Americans must undertake these enterprises are quite different from those existing elsewhere. Any works which we may construct must depend for their utilization and revenues on immigration, as they will be undertaken in an uninhabited country. In order to induce this immigration the people must be convinced of the benefits and utility of irrigation.

In India, on the contrary, irrigation works have frequently been undertaken in regions already overpopulated, and have produced a revenue by rendering the land sufficiently fertile and productive to support double the population which it was previously capable of sustaining. This has been accomplished in spite of the fact that the prejudices to be overcome and the difficulties encountered in inducing the people to make use of the water furnished, were probably quite as great as we will have to contend with in encouraging immigration to our now desolate wastes.

Administration.

In order to convey a clear understanding of the method of promoting irrigation development in India, it is essential first to give a brief outline of the present attitude of the English rulers toward irrigation.

At first the government permitted private corporations to construct and operate irrigation works; the earliest work planned by British engineers being undertaken by a private corporation on a guarantee of interest by the East India Company.

During the past thirty years the government has been very active in the promotion and construction of nearly all good works projected. These projects are studied, examined and reported on usually several times during a series of years, and when the government is finally satisfied with them, either as financial investments or as measures for the relief or prevention of famine, the work is sanctioned and the funds for its construction appropriated.

The works are administered by the Irrigation Branch of the Public Works Department of India, and in each of the several presidencies there is a separate head of this branch, who is the secretary to the government for that branch. All projects originated or reported on by the chief engineer of the presidency must be sanctioned and submitted by the secretary for the government to her Majesty's Secretary of State for India, from whom final authority to act must always emanate.

The government has absolute control over all waters and lands which

are British property, exclusive of native states or foreign possessions, and uses the water to the best advantage for the irrigation of the lands. After a work is constructed the government still retains control and management of all canals and branches, and distributes water into village or communal ditches. These latter are constructed and maintained by the village under supervision of the engineer.

Water rates are assessed on the area irrigated, and vary according to the nature of the crop raised, as this affects the amount of water which is required to mature it. The measurements are made by the canal officers, and the amount due from the different villages is assessed and recovered by the revenue collector of the district.

It is the duty of the Irrigation Branch to devise and estimate the cost and returns from irrigation works, as well as to construct and maintain them. The revenue branch levies and collects the taxes.

The head engineer of a presidency is the chief engineer. Under him are superintending engineers who have the supervision of a great canal system, which may be again divided into several divisions, each under the charge of an executive engineer. The latter have direct control and management of works and are assisted in constructive details by assistant engineers, under whom are upper subordinates, with sub-overseers under them again. The assistant engineer is most valuable in aiding in water distribution, while his first duty is in the execution of all works. Both he and the executive engineer have the powers of deputy magistrates for the protection of canal property.

The executive engineer is assisted in revenue and judicial matters by a native deputy magistrate and Zillahdars, under whom again are Ameens, who have charge of the water measurements and revenue collections as assistants to the Zillahdars. The charge and policing of the canals is under the Chokedars or constables.

The engineers are in the direct line of promotion by seniority and are recruited wholly from young men who have studied especially for the service at Coopers Hill College in England, or at Roorkee, or some similar college in India.

The positions of upper subordinates and sub-overseers are filled by young men who have studied at special schools maintained in India by the government, and are chiefly non-commissioned officers of the army, or are natives. The sub-overseers are almost all natives; there are also a few natives who are engineers.

Climate and Crops.

In India two crops are grown in each year; one is planted shortly after the other is harvested. The year is divided into two seasons or crops, the *Kharif* crop and the *Rabi* crop. The former is grown during the monsoon or rainy season, the latter in the dry season.

Though the average annual rainfall of the greater portion of India is very large, still nearly all of this occurs during the *Kharif* crop, and without the aid of irrigation only this one crop can be grown in each year. In the Punjab, Northwest Province, Bombay Presidency, and the larger portion of Central India, the rainfall varies in the lowlands from 20 to 40 inches per annum. Of this not more than 2 to 10 inches fall in the Rabi season. From the above considerations it will be seen that the conditions are very similar during the *Kharif* season to those existing in our eastern States where irrigation is not practiced. During the *Rabi* season the climatic conditions are quite similar to those of our arid regions.

It is as though India were a double country, one of which is humid and the other arid.

For these reasons the *Kharif* season will not be considered at all in the following, but the subject will be dealt with as though India had but one season or crop in the year, and that the rabi.

During the *Rabi* or autumn season the climate is very similar to that of our arid regions in the summer; the days are quite warm, the nights cool and pleasant, with no frosts excepting occasionally at the foot of the Himalayas, while rain seldom falls in this season.

Southern and Central India and the Sind—the Indus Valley—are in this dry season very similar climatologically to Arizona, Utah and the California Valley in the summer, while their agricultural products are much the same. The Northwest, Oudh and Punjab are like Idaho, Montana and Colorado, and in them the more hardy grains and vegetables are chiefly grown.

Owing to irrigation during the *Rabi* the land is capable of supporting nearly twice the population which it would otherwise sustain. In addition large tracts of heretofore uncultivated country have been rendered productive, and thus has the modern development of irrigation not only freed India from the dread of the fearful famines which used to decimate her population, but is enabling her to export in addition

such large quantities of grain that she is becoming one of America's chief competitors in the wheat markets of the world.

The difficulties attending the development of some of the larger enterprises which opened up new land, and the obstacles to inducing a sufficient immigration to these lands to render them profitable, are quite equal to any that will be encountered in the United States, though from a different cause. The trouble with us will be to find enough settlers to make the works pay during the first few years; while in India there is no lack of people, yet old and conservative customs and a fondness for the places in which families have lived for generations render it very difficult to overcome their prejudices and superstitions and to induce them to leave their ancient associations and tribes.

Irrigation Works.

The irrigation works of India are divided by the engineer into two great classes, Gravity irrigation and Lift irrigation. The former includes three main heads, viz.: Inundation canals, Perennial canals, and Storage works.

Lift irrigation is chiefly illustrated by wells. Of these there is little to say, though the area irrigated by them is considerable. They are in a country where labor is cheap, and are very useful adjuncts of gravity irrigation, utilizing the seepage water from the canals and irrigated fields which would otherwise be wasted. Owing to the cost of labor they will not be used extensively in the United States for many generations.

The three modes of raising water from wells are the *mot or churus* in the northwest provinces, Central India and Bombay; the *paecottah* in Bengal and the Persian wheel in the Punjab.

The mot consists of a leather bag made from a whole ox hide, which is lowered into the well by a rope passing over a pulley and when filled is raised by two bullocks walking away down an incline and drawing the rope after them. When the bag reaches the surface it is either emptied automatically or by an attendant. In Bombay the bullocks usually return by walking backward, elsewhere they are not trained to do this, and they face about up hill before returning. Sometimes two yokes of bullocks are used which are alternately hitched to the rope.

From the Calcutta *Engineer's Journal* it appears that two bullocks

working 10 hours a day for a season of ninety days will raise in this way 162 000 cubic feet of water or about 3¾ acre feet.

The paecottah or latha is the ordinary long wooden lever or well-sweep with a weight at one end and a bucket suspended from the other. It is worked by two men, and it is estimated that they will water thus about one-half acre a day, or three acres a season.

The Persian wheel consists of a large wheel revolving in a shallow well of water. This wheel carries a great number of buckets on its periphery, and these lift the water and spill it into a trough which empties into the ditches. The wheel is turned through a crude gearing by two bullocks which walk in a circle, turning a horizontal sweep. By this method 2 000 cubic feet of water per day may be raised. In the North-west Province alone, during 1888, there were in all 1 517 300 acres of land irrigated, of which 358 600 acres were irrigated by wells, the remainder by canals.

Inundation canals are used chiefly in the valley of the Indus, and as they have no permanent headworks they depend for their utility on two conditions, which are generally absent in the United States:

First.—The stream from which they are taken must flow at a high level relatively to the surface of the surrounding country, and its bed must be practically on the summit of a ridge, as is the case with the lower Sacramento and Mississippi Rivers.

Second.—The river must be subject to great annual floods lasting through a long period of time. Though these works have been among the most profitable in India, owing chiefly to their simplicity of construction, it will not be necessary to refer to them again, as America will probably have nothing similar.

The Indian financier divides the irrigation works into two great classes, called Productive works and Protective works.

Protective works are such as were constructed originally as a protection against famine, and have been sometimes called famine relief works. They have been built usually during famine times, in order, first, to give employment to the people so as to sustain them at the time; second, to furnish water as a protection against future famines. At present the majority of these famine works consists of storage reservoirs and tanks, which are located in the central and southern provinces. They have rarely been remunerative investments. The general cause of their unprofitableness in all probability lies not in the works themselves nor

in their uses, but in the conditions under which they are utilized; but it is not unlikely that similar works constructed in our West, under the more favorable conditions existing there, would prove as remunerative as the perennial canals of India have done.

It may be well to note that these perennial canals of the northern provinces, nearly all of which have returned regularly from 4 to 7 per cent. interest on the money invested, have been constructed in barren and uninhabited country, and that their success has been largely dependent on immigration.

We thus see that under conditions similar to those existing in America the works have been remunerative. On the other hand the protective works have all been undertaken in densely populated regions, where in ordinary seasons the crops are sufficiently good to support the people, but where in very dry seasons great loss of life is caused by the failure of the crops.

Because of this state of affairs the water stored in these protective works remains unused for a succession of years during which there is an abundance of rain followed by a dry season, when they are taxed to their utmost capacity to supply the demands made upon them. It is because of their long periods of idleness, during which no revenue is returned, that they fail to be productive. Of late years the people are beginning to use them more, and some works which were originally protective, and as such were an expense to the government, are now productive. Were these same works continuously used each year, as is necessary in America, they would be among the most remunerative in India.

The fact that the price of labor in India is so much less than with us is used as an argument that we cannot compare the value of their works with ours, yet with the increased amount of work done by an American laborer compared with that of a Hindoo coolie, and with the aid of our many mechanical operations as against their manual labor, the discrepancy in cost is not great. Labor costs, for women, generally four cents per day; men, eight to ten cents. Skilled masons get from eighteen to twenty-two cents per day, and carpenters and blacksmiths a little less.

At Bhatgur Dam the uncoursed rubble masonry work cost $1.75 per cubic yard, and at Tansa it cost $2.50 per cubic yard. At Betwa reservoir earth excavation in canal cuts as deep as 40 feet costs six and two-third cents per cubic yard, while surface excavation, with short haul, costs two and one-half cents per cubic yard. Uncoursed masonry

costs $2 per cubic yard, while coursed rubble costs $5.75 and ashlar about $6. At the same place brickwork costs $2.50 per cubic yard.

The returns derived from irrigation works in the two countries are more nearly equalized from the fact that we can impose a much higher tax for the use of water, or a higher water rate, than it is possible to demand of the poor farmers of India where from 2 to 5 acres support a large family.

From the revenue report of the Punjab the water in the canals appears to have done the following duty per second-foot, entering at the head, viz.: It irrigated from 70 to 150 acres, and produced a water rate or revenue of from $40 to $100. The duty on the supply utilized was 90 to 200 acres, and from $70 to $125 per second-foot, while the water rate charged per acre irrigated was from seventy cents to $1.15.

In the central arid regions in America we are able to get a revenue of from $1.50 to $3 per acre irrigated, which is equivalent for a duty of 80 acres per second-foot, to from $120 to $240 per second-foot utilized. In California, where water is scarce and the crops valuable, the rate is often many times higher, giving a proportionately high duty per second-foot.

The areas of land under irrigation in the various presidencies in India are enormous. The following table gives a statement of a few of the important features of the principal canals in the Punjab:

PERENNIAL CANALS.

	Swat River.	Western Jumna.	Bari Doab.	Sirhind.	All Inundation Canals.	Totals.
Minimum discharge of river in second-feet....	1 172	4 067	1 698	3 591	43 331	53 859
Maximum discharge of canal in second-feet....	700	3 000	4 000	5 130	12 000	24 830
Gross area commanded in acres.....	125 000	2 000 000	1 500 000	5 425 000	4 500 000	13 550 000
Area irrigable in acres...	110 000	550 000	525 000	800 000	1 400 000	3 385 000
Estimated cost.....	$1 400 000	$4 375 000	$5 250 000	$15 125 000	$2 475 000	$28 625 000
Miles of main canals and distributaries.....	145	1 185	1 420	4 950	3 760	11 460

The above table is given merely as an example of the extent and immensity of the irrigation works in one province. Those in the

Northwest Province, Bengal, Sind and Madras are nearly of equal magnitude, besides which there are many smaller works in these and the other provinces.

The principal crops grown in the northern provinces are grains such as wheat, maize, barley, millet, peas and lentils; also indigo cotton, rice, sugar-cane and hay. The method of irrigating is similar to that pursued in this country, and varies with the crop, soil and slope of the fields.

The subject of water-logging and the consequent production of fevers, the rise of the level of the subsurface water, and the production or efflorescent salts, alkali or *reh*, are subjects all of which have greatly troubled the Indian engineers, as they do ours. Generally it has been found that where irrigation is practiced the level of the water in wells has risen slowly but perceptibly. Water-logging and consequent fevers occur rarely and only in special localities where the drainage is bad, and in such places there seems to be no remedy but the artificial drainage of the country. In low-lying regions having heavy close soils it is advantageous and profitable to carry on a system of drainage parallel with the irrigation.

Likewise in a few limited areas the presence of alkali has become so abundant as to kill all vegetation and render the soil barren. This also is due to lack of drainage and the consequent evaporation of water from the surface of the soil leaving the saline matter behind.

Perennial canals are taken chiefly from the great perennial rivers which issue from the Himalayas, from whose snows they receive a never-failing supply of water. These rivers are very large, such as the Indus, Sutlej, Jumna and Ganges, none of which discharge much less than 5 000 second-feet at their lowest periods, while some of them discharge many times that volume. At high flood they increase to from 300 000 to over 1 000 000 second-feet. Though no such streams as these exist in the arid region of America, still a few, such as the Columbia, Missouri, Rio Grande and Sacramento discharge large volumes of water, much of which may at some future time be utilized. The methods employed in diverting and using these great streams of India may be of interest to us.

The topography of the Punjab and Northwest Provinces at the foot of the Himalayas is very similar in many respects to that of the great California Valley, or of the Colorado plains at the foot of the Rockies,

while the rivers issue from the foot hills much as the Sacramento, Feather and Mokelumne do from the Sierras, or the Arkansas from the Rockies.

At the points of diversion of their waters these Indian rivers have much the same slopes as those of California, and may, in all of their characteristics of surrounding topography, be compared to them.

The whole of the great agricultural valley or plain on the southern slope of the Himalayas, extending 1 500 miles east and west, and 200 miles north and south, is drained by two main rivers, the Indus and the Ganges. The tributaries of these rivers, flowing southward out of the mountains, suddenly emerge on the open plain with a fall at first of from 10 to 20 feet per mile, which rapidly diminishes to a foot or two per mile.

In consequence of this latter low slope the canals, in order to get a sufficient grade to carry them to the summits of the *doabs*, or interfluves between the streams, must be taken from the rivers as soon as they emerge from the hills. The chief objection to this location of the head works is that the first score of miles of the canal has to encounter considerable cross or side drainage, calling for the construction of many aqueducts, superpassages, siphons and other expensive devices.

The general plan pursued is to locate the diversion weir, not at the narrowest part of the stream, but at a relatively wide place, the cause being the great volumes of flood water to be contended with; and the object is that, by having a wide water way, the height of the flood of water passing over the weir shall be as little as possible. These conditions do not exist with us to the same extent, and the same reasons for choosing a broad bed of the river for the weir site are not so binding.

The first consideration in designing a canal is the source of supply. This may be from a perennial stream or from storage reservoirs. In either case it will be necessary to make a thorough examination of the hydrography of this source in order to make certain of the quantity of water available. This being known, one of the most important considerations remaining in relation to the location of the head works is the point on the river's course at which the canal will be taken off. This must be high enough up on the river to command sufficient level, and if possible to tap the stream where the water is clear and not laden with silt.

In order to determine the quantity of water required the area of irrigable land must be ascertained and the duty of the water for the crops and soil determined. The duty of water is the area of land which a fixed unit of water will irrigate. In India, as in America, the unit adopted is one cubic foot per second of flow, though in our mining regions the miners' inch is still in common and popular use. When we say that the duty of water is 80 acres, we mean that one cubic foot per second, or as it is better expressed, a second-foot of water, flowing throughout the irrigation season, will irrigate 80 acres. In speaking of the volumes of flow of streams we say that they discharge so many second-feet. In speaking of the contents or capacities of storage reservoirs the Indian engineers quote them as holding so many cubic feet of water. This unit becomes cumbersome in dealing with the thousands of millions of cubic feet which a large reservoir will store. The practice of the engineers of the U. S. Geological Survey is to quote capacities in acre-feet—that is, the quantity of water necessary to cover an acre 1 foot in depth, or 43 560 cubic feet.

We may speak of the duty of water by saying that so many acre-feet or fractions thereof are necessary to irrigate an acre. Thus in Colorado between 1 and 1½ and 2 acre-feet are necessary to irrigate an acre. The duty of a second-foot of water may be spoken of in financial terms by saying that it is worth so many dollars. Thus by irrigating a given number of acres it will produce a revenue of that many dollars.

The duty of water differs very widely, and is as yet but roughly known for the various sections of our own country. In the central arid regions it varies according to circumstances from 50 to 100 acres irrigated per second-foot.

In designing canals in India the expenditure of water is sometimes reckoned per linear mile of canal, and this has been found to be from 6 to 8 cubic feet per mile. Such a method of computation is only possible when the main and branch lines of canal have been determined previously, when it is very convenient for calculation, as it enables the engineer to regulate the cross-section of the canal along its entire length, diminishing it as the water is expended.

It is customary in India to try and give a proportion of width to depth of the channel corresponding to that which the natural streams in a given locality have created for themselves. In very wide canals this varies usually from one in thirteen to one in fifteen ; in smaller canals,

however, this ratio will be greatly reduced. A rule used on the Soane canals for determining the ratio of width to depth is to make the bottom width equal to the depth plus one squared.

The slope of the sides of the canal will differ greatly and be dependent chiefly on the soil in which it is excavated. The slope of the bed of the canal is also a variable quantity, likewise depending largely on the soil. If too great the bed of the canal will be eroded; if too small silt may be deposited, weeds grow and clog the water way, and in order to carry a given quantity of water, the cross-section of the canal, and hence its cost, be increased as its slope diminishes. Accordingly it is considered best to err on the safe side and give the greatest slope apparently desirable. In case too great a slope has been given, or the country falls too rapidly, this may be remedied by introducing falls or rapids in the channel to diminish the general slope, and so concentrate the loss of grade in a few places.

The minimum velocity given in India, sufficient to prevent the growth of weeds and the deposit of silt, is about 1½ feet per second. This is too low according to American experience, and we usually consider that in our climate and soils from 2½ to 3½ feet per second is a minimum. The maximum varies greatly with the soil, though in ordinary earth, not gravel, it will seldom exceed 4 feet per second.

In a report, Major Crofton, R.E., late Inspector General for irrigation in India, says: "In very sandy soil a mean velocity of 2.5 feet is best; in light and sandy soil 3 feet; while heavier soils, if sandy, will stand 3.5 feet per second."

Most of the larger canals of India were designed for navigation as well as irrigation, but since the introduction of good wagon roads and of railways, the necessity of using canals for purposes of inland traffic has greatly diminished, and it is now generally considered not advisable to make the canals navigable. There are several reasons for this: a navigable canal is far more expensive to construct and maintain, since owing to its low grade the cross-section must be made large in order to carry the water needed. Then the bridges must be made higher, and the drainage works required are of the more expensive kind, such as aqueducts and superpassages instead of simpler constructions. Moreover, it is wasteful of water, for a certain large volume must flow through the canal for purposes of navigation only. Owing to the light grade necessary on navigable canals much silt is deposited and the growth of weeds encouraged, adding to the cost of maintenance.

The banks of navigable canals are generally cut for laterals or distributaries as seldom as possible. Accordingly smaller parallel canals are constructed on either side, tapping the main one at long distances, while smaller distributaries are taken from these at convenient places. One reason for this is to avoid injuring the banks of the main canal by too many breaks, but the principal reason is because the fall of the navigable canals being very small, the upper end of a level will be too far below the surface of the surrounding country to permit laterals to be diverted, while the parallel canals can be kept near or above the surface.

Diversion Weirs.

The diversion weir at the head of a canal must be high enough to raise the water at low stage into the canal. In India these weirs are generally from 10 to 15 feet above mean water, though in some cases they are much higher, notably the weir at Vir on the Nira River, which is 43 feet in height. The canal should start out as nearly as possible at right angles to the course of the river, and at its head there must be a set of regulating gates to control the amount of water admitted. A set of flushing sluices is inserted in the weir at the end adjacent to the canal head, giving a free sweep of the stream past the mouth of the canal to prevent the deposit of silt at its entrance. The sluices are furnished with gates which can be closed when the river is low in order to raise the water to the height of the top of the weir.

The diversion weir is always constructed of masonry, and is usually intended to permit flood waters to pass over it, though in some special cases the level of the water is raised by means of a weir that is practically a long set of sluice gates extending the whole width of the river. In times of flood these can be raised high above the stream or removed, thus permitting the free flow of the water. The reason for this form of construction is that the grade of the stream is very low at the point where it is dammed. In flood an ordinary weir would so raise the height of the river as to submerge the country and cities above it.

Diversion weirs in India may or may not be open. The advantage of the closed or solid weir is that it is self-acting, and if well made costs little for repairs or maintenance. Its first cost is, however, greater than that of an open weir, and it interferes with the regimen of the river, causing deposits of silt above it, and perhaps causing the river to seek another channel. With the open or sluice gate weir interference with

the normal action of the river is reduced to a minimum. The scour produced by opening the gates prevents the deposit of silt, and its first cost is less than that of a closed weir.

Indian experience has proven that a deep and firm foundation for low diversion weirs over which great floods may pass, is unnecessary to insure their integrity. In several cases, notably weirs at the head of the Agra, Lower Ganges and Soane canals, where the depth of the sand in the river bed is great, the weirs are founded on "wells" or coffer dams made of brick or masonry, sunk in the sand to a depth of 10 to 20 feet or more, and on these the superstructure has been erected.

As an example of well sinking, those used for the foundations of the Nadrai Aqueduct on the Lower Ganges Canal were sunk to an average depth of 56 feet. There were 102 wells, 20 feet in external diameter, and 166 wells, 12 feet in diameter. Great care had to be used in excavating the sand from the interior of these wells in order to cause them to sink vertically, and this was done chiefly by dredging from either side according as the well got out of perpendicular. Sometimes, and especially in clay, the wells got stuck and failed to sink, when it became necessary to load them with railroad iron or a temporary superstructure. To test their resistance against subsidence, some of the wells, after they were hearted with concrete, were loaded with a great temporary superstructure of bricks, which in the case of one well weighed 1 541 tons when the total subsidence was but 1.1 inches.

Wells and blocks are built in single or double rows, the distance between wells being 2 or 3 feet only, while between rows it may be several yards. They are from 10 to 20 feet in exterior diameter, the former cylindrical in form, the latter rectangular, with walls from 2 to 4 feet in thickness, and are shod with an iron cutting edge. They are built up as the sand is excavated from their interiors and as rapidly as they sink. When completed they are usually hearted with rubble or cement, and the spaces between the wells or blocks in any row are filled with cement or dry stone packing, and on this artificial foundation the weir or other superstructure is erected.

Weirs in rivers having great flood discharges are sometimes made very low and flat, with an up stream slope of from 1 on 10 to 1 on 16, and are constructed with two or three central longitudinal walls of masonry extending the entire width of the river, and between these loose rock filling is placed.

The Soane weir at Dehree is 12 470 feet long and 19.3 feet high. It consists of three parallel walls of masonry running across the river and founded on wells, and between these walls is dry rock packing. The up stream slope is 1 on 3 while the down stream face is 1 on 12. The height of this weir is increased by placing on its top a row of iron shutters each 18 feet long and 22 inches high. In time of flood these fall automatically. In great flood this river discharges 1 200 000 second-feet of water over the weir. The height of the weir above the sill is but 8 feet, and its width on top is 5 feet.

A peculiar weir is that at the head of the Lower Ganges Canal at Narora. It is constructed entirely of brick built into walls and founded on wells. The tail below the brick apron is dry packed rock. The weir is 7 feet wide on top and the wells are sunk but 6 feet below the river bed. The cross-section of the weir is rectangular, allowing a vertical drop of 9½ feet on the down stream side to a cement floor founded on wells. The total length of the masonry and pitched flooring below the weir is 146 feet.

On boulder foundations a different weir is erected, and it may be given a much steeper slope on the down stream face. In one notable case, that of the head of the Ganges Canal at Hurdwar, a temporary boulder dam of loose stones is built up each year, as it is washed away annually by the floods. This weir consists really of three parallel weirs across the channel, placed one below the other, so that the water leaking through the loose round stones of which they are composed is caught by the weirs below.

Where a good rock foundation is to be had near the surface the weir is given an entirely different cross-section, and is constructed of masonry. If the overfall height is not great it may be nearly vertical up stream and will have a slight slope of about 4 on 1 down stream. Or it may be sloped up stream and be vertical down stream, thus giving a clear overfall for the flood waters, which will then fall on a water cushion formed by a subsidiary weir placed below the main weir.

The flushing sluices placed in an open weir consist of a number of masonry piers placed at regular intervals apart, and grooved for the reception of planks, needles or gates by which the water can be kept under control. The intervals between the piers are from 6 to 12 feet, while the depth of the gate is equal to the height of the weir. A masonry flooring is carried across the entire width of the sluice-way, flush

NADRAI AQUEDUCT.

Nadrai Aqueduct over Kali Nadi Torrent. General view sinking "wells" for foundations of piers. In foreground wells for a pier have been sunk to full depth, 56 feet below ground surface. In mid distance a well 20 feet diameter loaded with a temporary superstructure of 1 541 tons of bricks to test resistance to subsidence. Beyond it a group of four 12-foot wells loaded with 2 172 tons of brick to test them. In distance, sinking of wells in progress.

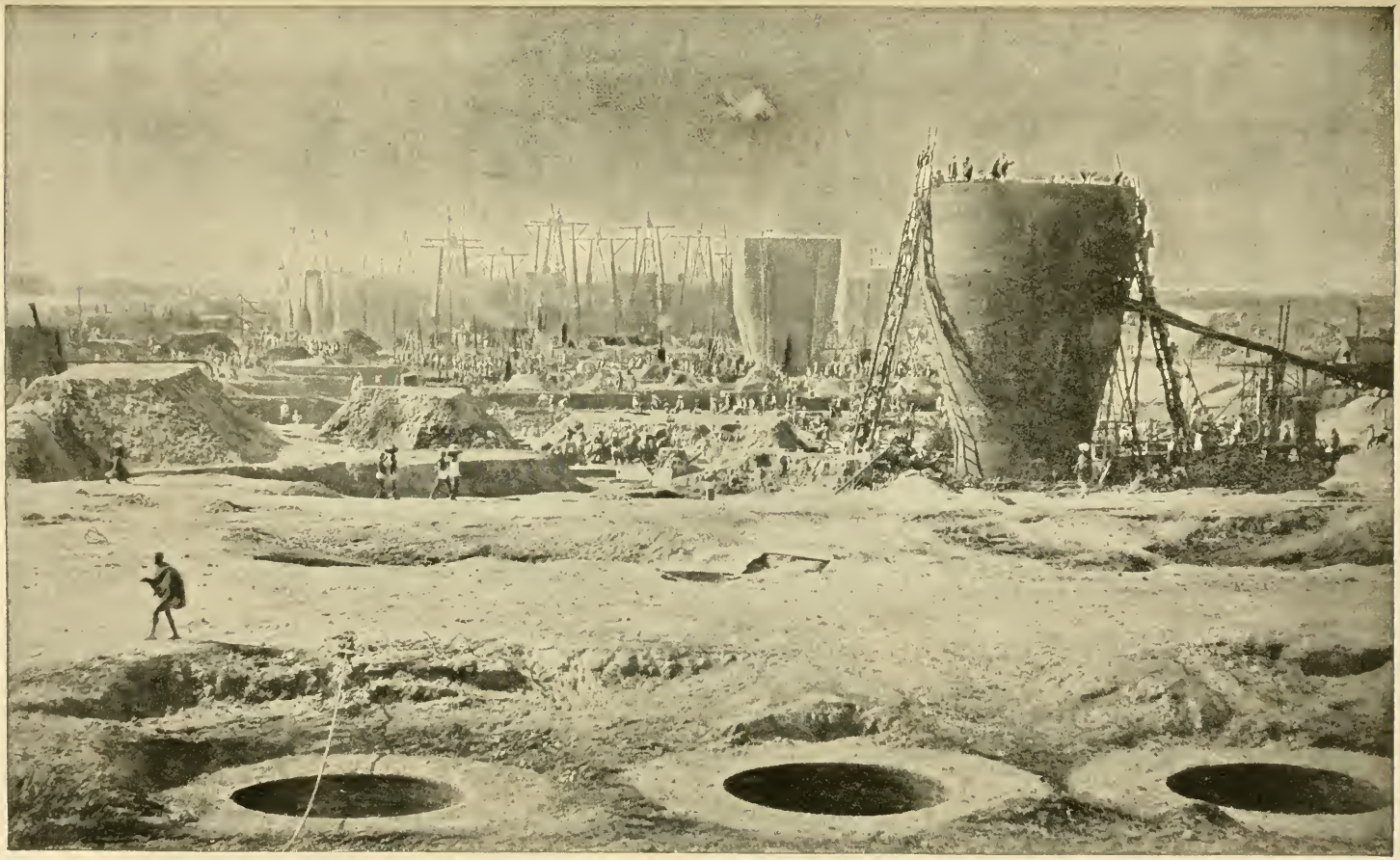


PLATE LXII. NADRAI AQUEDUCT. Vol. XXIII, p. 232.

with the river bed, and is carried well into the bank of the river, which is protected by a curtain wall of masonry up and down stream.

In the case of rivers subject to sudden floods it is well to have drop gates which fall either automatically or with little labor. These gates are usually hinged to the flooring of the sluice-way and are held up by chains attached to the piers. In the great weir at the head of the Soane Canal at Dehree there are two sets of gates. The down stream gates are hinged to the floor by an iron strut which is hinged to the up stream side of the gate below its center. When dropped they fall into a water cushion which eases the shock. The up stream gates are hinged at their bottoms and fall up stream. When raised the force of the shock caused by the sudden pressure of the water behind is relieved by a set of pistons working in cylinders containing water and having five holes each. When all are open and it is desired to close the gates, the up stream gates are first raised; this removes the water pressure from the down stream gate, which can then be raised. Then the up stream gate is lowered again, and when a flood rises to the top of the lower gate the latter falls automatically.

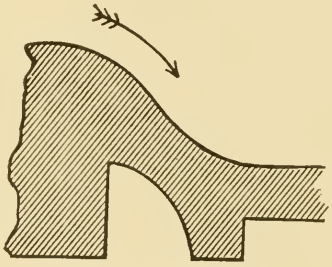
At the head of the canal a set of regulating gates must be constructed large enough to admit as much water as the canal will carry, and by means of which the amount of water admitted to the canal can be controlled. These regulators consist, like the sluice-way, of a set of grooved piers resting on a firm foundation carried across the canal bed. Flood or automatic drop gates are not necessary, but the water admitted may be controlled by a set of simple planks or by a gate raised and lowered vertically. These gates are usually raised by a windlass and chain, but in some cases, where under considerable pressure, they are raised and lowered by a hand lever working a screw.

In the case of streams which carry much sediment in suspension, and also in order to establish complete control over the water after it enters the canal, it is necessary to construct escapes or flushing channels at intervals along the line of the canal. These escapes are to a canal what safety valves are to a steam boiler, and on most canals are provided at intervals of from 30 to 40 miles along its course. The first and main escape is placed about half a mile or more below the canal head, and should be closed with a set of regulating gates. When it is desirable to scour out the silt which has accumulated in the canal above the escape, the latter is opened and a large head of water is let into the canal and

runs off through the escape, thus cleaning out the canal above. It is usual in such cases to decrease the slope of the canal for a short distance below its head in order to cause the deposition of matter carried in suspension between the head and the escape.

The escapes along the line of the canal permit the discharge of local drainage which has been allowed to enter, and make it possible to empty the canal rapidly when it is necessary to repair or clean it. These escapes should also be provided with head or regulating gates, and should have a capacity sufficient to discharge the whole volume of the canal rapidly, and to do this must have a much steeper slope than the canal.

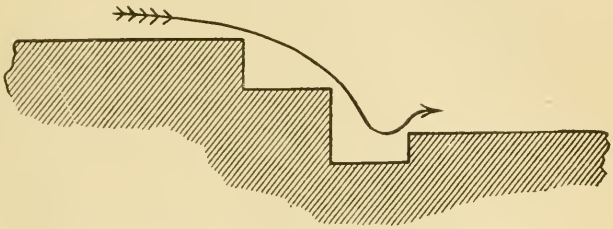
As the natural fall of the country through which a canal runs is usually greater than the slope of the canal, it becomes necessary to compensate for this difference of slope, and this is done by concentrating it in a few points where vertical falls or rapids are introduced. Their location is usually decided by the place where the canal becomes too high above the surface of the ground, and in India their exact position is usually made to coincide with that of a bridge or similar work in order to economize masonry construction.



Ogee Falls Cross-Section.

Ogee falls were adopted first on the Ganges Canal, and are of the shape shown by the diagram, the intention of the design being to break the force of the fall. They were about 8 feet high, but the shock of the fall was so great that they have been modified so as to give two vertical falls, the lower of which drops into a water cushion, greatly diminishing the shock. On some canals the action is still lessened by making the water play over a wooden grating, which reduces the shock by dividing the stream into a number of fine threads. These falls are invariably constructed of masonry laid on a deep and firm foun-

dation, the banks are protected by masonry and paving for some distance below the fall as is also the bed of the canal, and the flooring terminates in a row of sheet piling. The fall is divided usually into



CROSS-SECTION, VERTICAL FALL WITH WATER CUSHION.

longitudinal compartments, so that when repairs are necessary one portion can be laid bare at a time.

The effect of the fall is to hasten the velocity of the water in the canal above it, and as this produces dangerous scour it has been found necessary to head up the water above the fall by raising a low weir on its crest.

Instead of falls, rapids have been constructed with success on the Bari Doab Canal. The slope of the rapid is paved with loose boulders which are confined by walls of masonry at intervals of 40 feet longitudinally and across stream. Dry boulder or rock is not to be depended on for velocities over 15 feet per second, and the slope must be modified accordingly or masonry used. The result of experience shows that vertical falls with wooden gratings and water cushions are the best form yet tried. On smaller canals and distributaries the falls are generally into masonry tanks, the bottom of which is below the bed of the lower canal level forming a water cushion.

DRAINAGE WORKS.

In canals flowing along the sides of hills or slopes, great difficulties are encountered and expense incurred in diverting or passing the side drainage which crosses its line. Much can be done by diverting water courses or constructing short drainage cuts emptying into natural drainage lines.

When the drainage cannot be diverted it can be passed in one of three ways. If the stream is at a lower level than the canal, the latter may be passed over it in an aqueduct. If at the same level, it may be permitted to enter the canal with proper regulating gates below, and an escape

opposite it. If the stream is at a higher level than the canal, it is generally carried over the canal in an aqueduct which is called a superpassage. Modifications of these are when the canal is carried under the stream, or the stream under the canal in an inverted siphon. And in some cases a combined siphon-aqueduct has been constructed.

The most notable aqueduct is that which carries the Ganges Canal over the Solani Valley at Roorkee. This consists of an earth embankment 2¾ miles long across the valley, about 16½ feet high, 350 feet wide at the base and 290 feet wide on top. On this the canal banks are formed, being 30 feet wide and 12 feet deep, lined for their whole length with masonry or stone paving. The aqueduct which crosses the river is of solid masonry, 920 feet long, with a clear water way of 175 feet in fifteen spans of 50 feet each.

Another great aqueduct is that which passes the Lower Ganges Canal over the Kali Nadi Torrent at Nadrai, of which a description of the well sinking for the foundation has already been given. This aqueduct carries 3 175 second-feet of water, and consists of fifteen masonry spans of 60 feet each; the maximum pressure on the foundation walls of the pier does not exceed 2½ tons per square foot. The total superficial water-way is 23 430 square feet, which is at the rate of 9.8 square miles of catchment area, and will pass a flood of 140 000 second-feet, having a mean velocity through the aqueduct of 60 feet per second. Many smaller aqueducts are to be found on the canals everywhere in India, and they are universally constructed of masonry in the most substantial manner.

It is common to admit the waters of small drainage channels directly into the canal when they meet at the same level. In cases of large streams, as happens with the Rutmoo Torrent, which enters the Ganges Canal at level, carrying a large volume of water charged with sediment, special provision must be made. In this case the inlet is plain, but in the opposite canal bank is an escape controlled by regulators, and below it is a regulator in the canal to permit the proper amount of water to flow down it.

In the construction of a superpassage special care must be taken to give it the proper slope, so that it shall not become choked with sediment, and the masonry and reveting must be carried far enough to either side of the canal, up and down the torrent, to protect the foundations. Some superpassages which are half sunk in the ground become in time

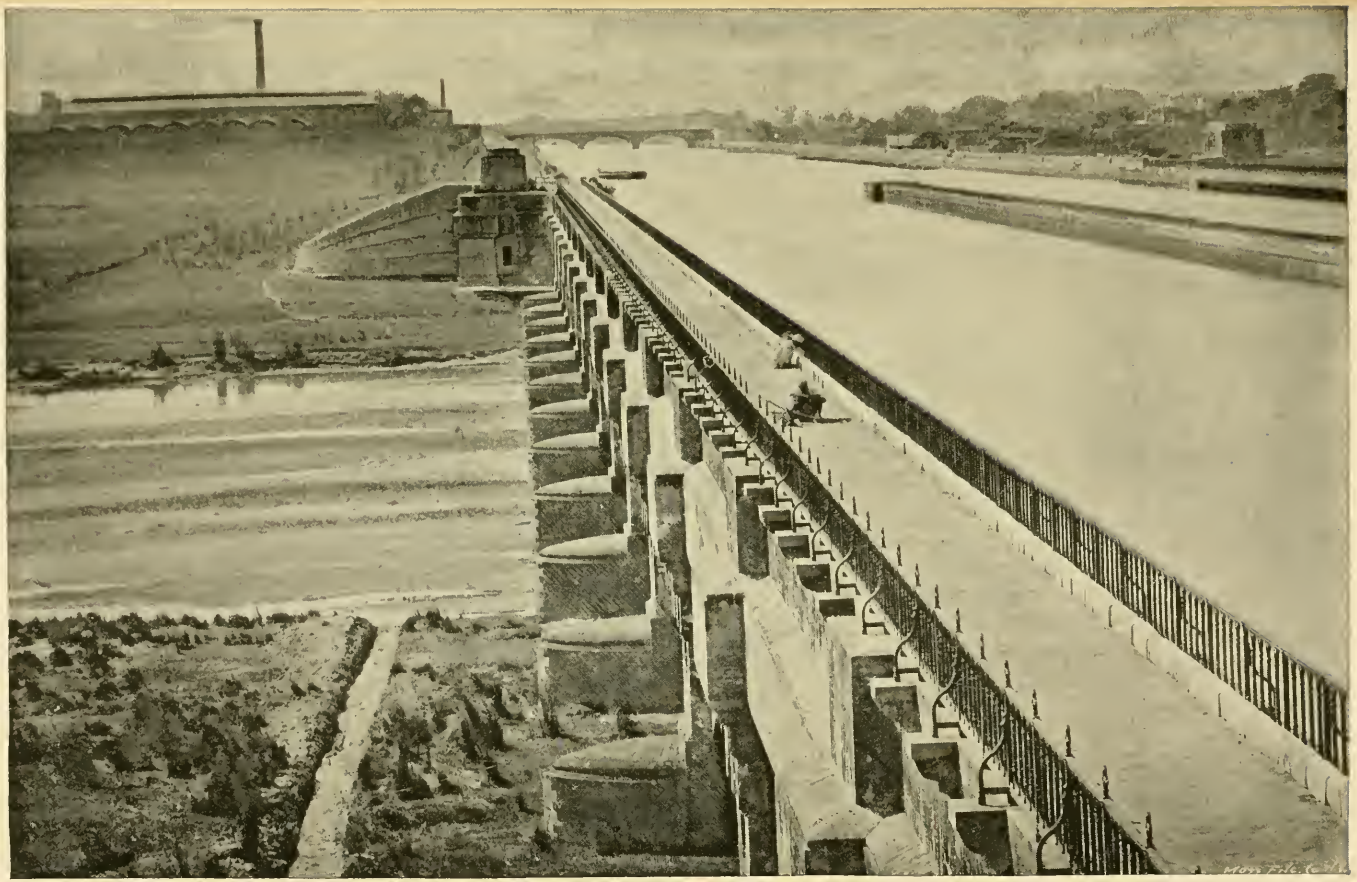
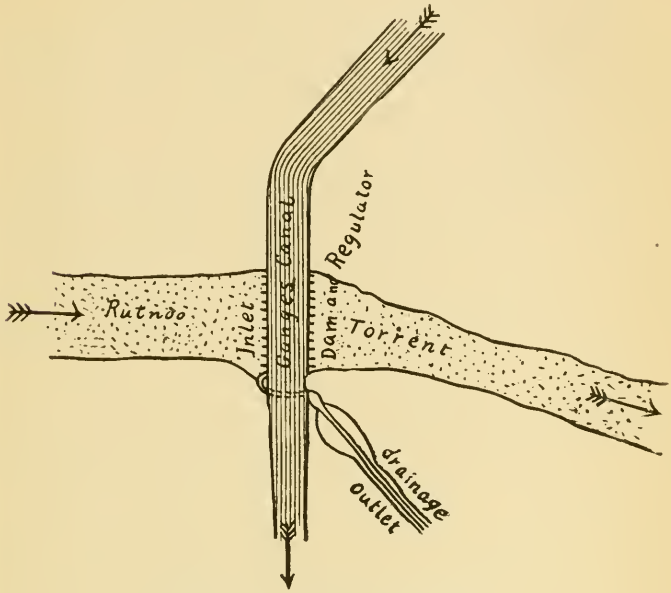


PLATE LXIII. SOLANO AQUEDUCT AND VALLEY SEEN FROM ABOVE. Vol. XXIII, p. 236.

of flood semi-inverted siphons; such is the case of several small superpassages on the Nira Canal, and of the Kao Nulla siphon-aqueduct on the Soane System, and of several on the Sirhind Canal.



GANGES CANAL, RUTMOO LEVEL CROSSING.

DISTRIBUTION OF WATER.

Distributaries, or laterals, are small branch canals or ditches which have a masonry, wooden or iron regulator at the head. From these the farmers take their water courses to their fields. Like the main canals these distributaries are laid out by and are under the control of the canal engineers, and answer the purpose of a system of storage reservoirs to the private or village channels.

In order to best serve their purpose, the distributaries should follow the water-shed of the country, and to this end a drainage and contour map is first constructed. The capacity of the distributary which traverses each separate drainage divide is then proportioned to the duty which it has to perform, the bounding streams limiting the area which each distributary has to irrigate. The accompanying map shows the method of planning a system of canal distributaries.

It is customary to take off the water of a distributary near the surface of the canal in order to obtain that which is clearest, or charged with the least amount of silt, and to keep the bed of the distributary at a sufficiently high level to permit surface irrigation throughout its entire length. To this end the bed of the distributary is generally a few feet higher than that of the main canal. In America water charged with sediment is generally preferred to that which is clear, from the belief that it aids in fertilizing the soil.

According to Major Brownlow, R.E., the greater the amount discharged by a distributary the smaller will be the proportionate cost of maintenance. For a channel 12 feet in width discharges more than double that of two channels each 6 feet wide; and the cost of patrolling and repairs to bank will be half that of the two smaller ones. Experience proves that irrigation can be most profitably carried on from channels 18 feet wide at bottom, and with about 4 feet depth of water. On the Eastern Jumna Canal, during 1856–60, the expenditure on all distributaries of 12 feet head width and upwards was .123 of the revenue, while on all those below 12 feet the cost of maintenance was .223 of the revenue, or nearly double the first. The relative value of a cubic foot per annum from the same experiments on channels of respectively 12, 6 and 3 feet in width was as 10 : 7 : 4. The increased action of absorption in small channels, with diminished volumes and velocities, accounts for the difference. The depth of water should seldom be less than 4 feet, and the engineers endeavor to keep the surface of the water at from 1 to 1.5 feet above the surface of the country, in order to secure gravity irrigation. Indian engineers condemn the practice of raising the water in the channels to the surface of the country by means of dams or stops, made by introducing planks in grooves made for the purpose. They say that this converts a freely flowing stream into a series of stagnant pools, causing the growth of weeds, the deposit of silt, and unhealthy condition of the neighborhood, and is, moreover, very wasteful of water, as much is thus wasted by absorption and loss of head.

The water for the village ditches is taken from the distributary by means of hollow wooden or iron pipes let into the banks of the canal flush with its bed, and are stopped at the outlet by a simple valve or plug.

While the water rate is at present charged in India according to the area irrigated and the crop raised, it is conceded that it would be much

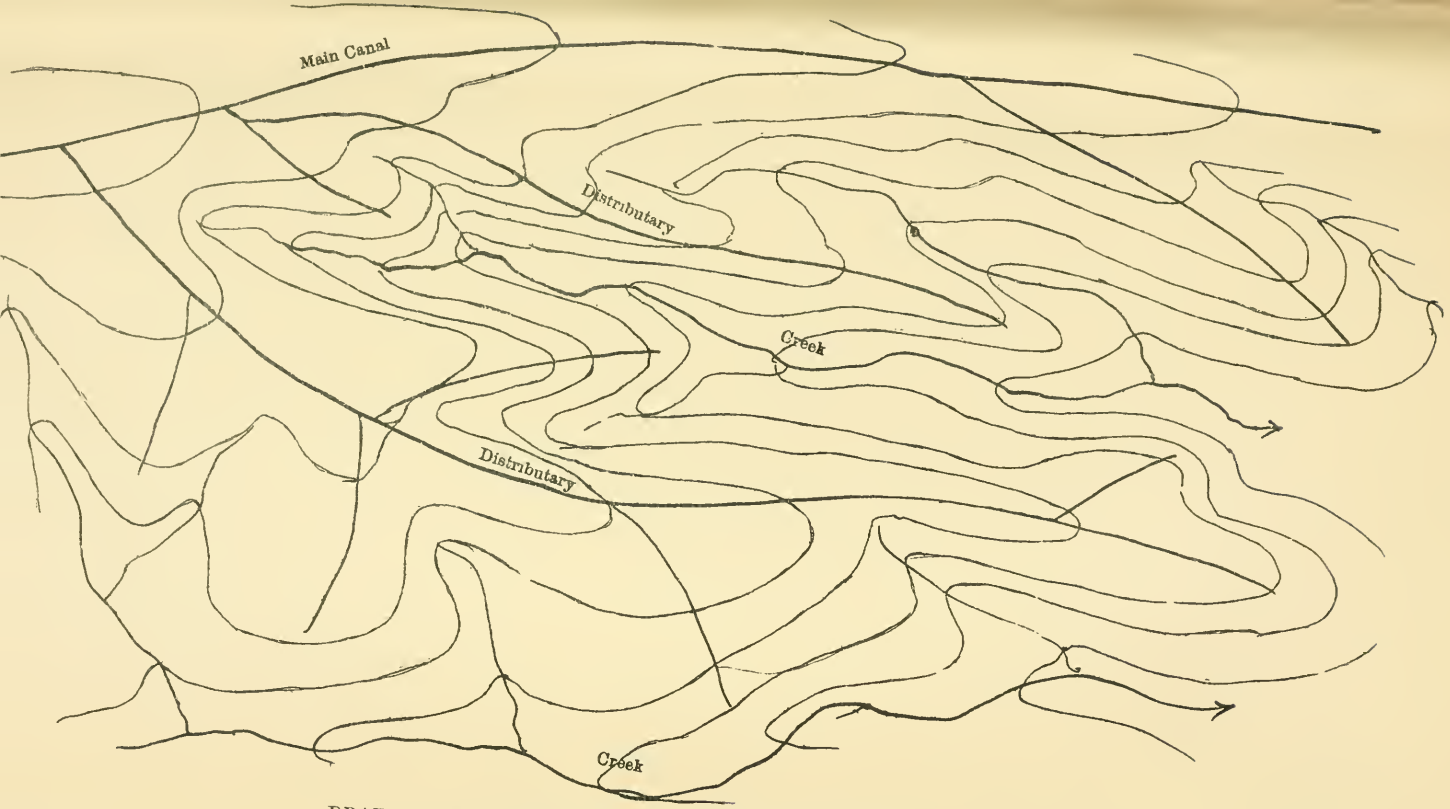


PLATE LXIV

DRAINAGE MAP SHOWING ARRANGEMENT OF DISTRIBUTARIES.

fairer to charge for the water according to the quantity used, as it can make no difference to the canal proprietor what becomes of the water after he has delivered and been paid for it. The difficulties in the way of delivering water by accurate measurement have heretofore been insuperable, chiefly because no practical method of measuring water under a constantly varying head has yet been hit upon.

On all the older established canal systems, especially when the supply entering the head of the canal is insufficient for the demand, a system called *tateels* has been established. This consists in regulating the amount of water given to the various irrigators by closing their outlets for successive periods of time in regular rotation, each man being compelled to take the water when his turn arrives or else lose it. It is best to impose tateels on long portions of the distributory at once. Short ones have very little effect in forcing the water down to the tail of the canal. Outlets are closed in the first length for four days, on the second portion for three days, and so on, and then this order may be reversed, having regular periods of rotation.

Great Canals Constructed.

Little further can be said of typical canal construction in India other than to give an outline of the magnitude of some of the more important works. Details of aqueducts, superpassages, regulators and weirs have been given already in their separate places.

The greatest of all these works is the Ganges Canal, which is diverted from the Ganges River at Hurdwar, at which place it emerges on to the broad northern plain where the country has a general slope of about 20 feet per mile. Its head works are peculiar, the discharge of the river being much greater than the amount of water needed, and the current swift, while the bed is composed of boulders to a great depth. It was deemed advisable to divert the water by means of a temporary boulder dam into the Hurdwar Channel, and this is kept open and separate from the remainder of the stream by an elaborate system of river training works.

These temporary weirs—for there are three of them, one back of the other—are destroyed annually by the floods and rebuilt each year, this method being cheaper than to construct a permanent weir. At the canal head are a set of regulating gates, adjacent to which is a permanent weir with flushing sluices which close the Hurdwar Channel.

The slope of the canal bed is at first 15 inches per mile, and the velocity of the water 3½ feet per second, which is too high and constantly erodes the canal banks. The canal carries 6 700 second-feet of water at its head, where the channel is 170 feet wide at bottom, with 8 feet depth of water. Owing to the great slope of the country in the first 30 miles of the canal many expensive falls have had to be introduced, and great aqueducts and superpassages are necessary in order to pass the local drainage.

This canal controls 10 440 000 acres, of which it irrigates 1 000 000 acres, and has 456 miles of main canal, 2 599 miles of distributaries, 895 miles of escapes and drainage cuts, and its total mileage is 3 910. Each distributary would be a large canal in America, and has numerous masonry bridges, regulators and aqueducts along its line.

Another great canal system is that of the Sirhind Canal in the Punjab. This canal is diverted from the Sutlej River at Rupar, where the river emerges from the Himalayas to the plain. The diversion works consist of the usual weir with undersluices in it, and regulators at the canal head. A few miles below the head is an escape into the Sutlej, which is very efficient in removing deposits of silt. In the first few miles the fall is great, and there are a number of drainage works, of which the Siswan Superpassage carries 20 000 second-feet and the Budki Superpassage 30 000 second feet of water.

The main canal and principal branches are in all 4 950 miles in length of which 503 miles are main navigable canal. Its discharge is estimated as 7 000 second-feet, though it generally carries only from 3 700 to 4 000 second-feet. The minimum discharge of the Sutlej River is often as low as 3 500 second-feet, while its maximum flood discharge may be 100 000 second-feet. The rainfall over the irrigable area varies from 10 to 35 inches per annum, and during the autumn crop it is from 1 to 6 inches only.

The gross area commanded by the canal is 4 521 000 acres, of which 800 000 acres are irrigable. During the autumn of 1889, only 338 000 acres were irrigated, of which 38 000 acres were double cropped. In the autumn of 1889 the duty of water per second-foot on the supply entering at the canal head was 119 acres or $113. On the supply utilized it was 127 acres or $121, and the average water rate per acre irrigated was 95 cents. The percentages of the principal crops irrigated were as follows: sugar cane, .33; maize, 6.33; cotton, 3.36; wheat, 43.63; barley, 3.66; millet and small grains, 16.10; and all other crops 26.59.

PLATE LXV. GANGES CANAL—RANIPUR SUPERPASSAGE. Vol. XXIII, p. 240.

Let us see what this canal would have cost to construct and maintain in the United States. In making such a comparison the amounts expended for pensions, furloughs, navigation works, etc., must be deducted, and due allowance made for the difference in cost for each class of construction in the two countries. The total expenditure on this canal to 1889 for construction, maintenance and repairs was $15 350 000, of which $3 100 000 was interest on the outlay while the work was under construction, and of the whole $500 000 was leave and pension allowance to employees. The total original cost of the works was $12 000 000, of which, exclusive of cost of land and maintenance, $400 000 was for headworks, $6 500 000 for the main canal and branches, exclusive of $1 100 000 for right of way and navigation works, and $300 000 was for drainage and protective works.

The office establishment, less pensions—and this establishment is a very expensive one in India—cost $2 200 000, and the tools and plant $1 100 000. The earthwork alone on the main branches cost $2 600 000, and on the distributaries $800 000, or in all $3 400 000. For 800 000 acres of irrigable land the earthwork cost $4 25 per acre; or, our contract prices in the West being say 10 cents per cubic yard, while theirs is but four cents—this earthwork would have cost us $10.62 per acre. The masonry works, such as falls, weirs, regulators and bridges cost in all $2 400 000, or $3 per acre irrigated. Rubble masonry costs in India about $3 per cubic yard. In our West it averages say $6. Hence these masonry works would have cost us $6 per acre.

In our works we would avoid the expense of the numerous masonry bridges constructed in India. Again, we would do little masonry construction, but would use much iron and wood, which with us are far cheaper. The cost of these items would accordingly be proportionally much less in America. Say one-third can be deducted for the cheaper material and less masonry work, and it would cost us $4 per acre irrigated. Drainage works cost about seventy-three cents more per acre, or say in all the Sirhind Canal would have cost us $15.35 per acre irrigated, against $8 in India.

As the Sirhind Canal is a new work, at present only 476 000 acres are irrigated from it of 800 000 estimated as being irrigable. This, however, pays a profit on the capital invested of from 2 to 4 per cent. It should yield from 6 to 8 per cent. when worked to its full capacity. The water rates charged average ninety cents per acre irrigated; we would charge

at least $2; in some localities far more. As the cost of construction in America would be twice what it was in India, while the receipts per acre are more than double those of India, it is not improbable that under similar circumstances such a work, when partially utilized, would yield us 5 per cent., and when doing its maximum duty would realize 10 per cent. net profit on the capital invested, besides furnishing homes for many thousands of people, and largely increasing the aggregate wealth of the country.

The chief inducement to canal enterprise in America has not been mentioned, because it does not exist in India. This is the increment in value of the land served by the canal. There is no such increment available to private enterprise in India, because the Government is the sole owner of the land. Here, however, where land can be purchased for from $1.25 to $2.50 per acre, and when irrigated will sell for from $40 to $100 per acre, or bring an equivalent revenue, the increased return on an investment is obvious.

Storage Works.

Storage works in India are usually divided into two general classes, reservoirs and tanks. The former are generally distinguished by being very deep, are closed by masonry dams, and are on running or flood streams. The latter usually are shallower, have earth dams or dams composed of earth with a masonry core or face, and are on plains or interfluves above the drainage lines, or else on small streams, the flood discharges of which are so small that all the water can be stored or easily wasted. Tanks may be filled by canals diverted from some large stream for that purpose.

In deciding on the location of any storage work many different considerations have to be discussed and weighed. It should be placed at a sufficient elevation above the lands which it is intended to irrigate to permit of the water being taken to them by means of natural flow. The storage work may be so situated that it is immediately adjacent to these lands, and a very short canal line will command them. Or it may be at some distance from them, requiring a long line of canal. Again, the storage water, when wanted, may be turned back into the natural drainage channel through which it will flow to the neighborhood of the lands where it will be again diverted. The second case may be very expensive if the canal line is long, but the last is most expensive and wasteful, as

the loss by absorption and evaporation is much greater than in a well constructed canal, while there will be added the additional expense of a weir at the point where the water is again diverted from the natural channel.

Knowing the area and location of the irrigable lands, the first consideration is the location of the storage site. This may be discovered after much seeking, and in order to be desirable it must be of such a character that the cost of the construction of the works will not be prohibitory.

The next consideration is the source of water supply from which to fill it. This may be either from an intermittent stream, or if from a perennial one its discharge must generally be small, otherwise storage would be unnecessary. A thorough hydrographic survey of the catchment basin above the reservoir is necessary in order to discover the maximum and minimum amounts of precipitation, percentage and rate of run-off, and the total quantity of water available for storage. Steam gaugings must be maintained for a few years, and all possible efforts made to discover the volumes of maximum floods as well as total quantities of water discharged. A good topographical survey is usually first executed, as from the maps thus constructed the relation between irrigable land and reservoir site, and the latter and the catchment areas, can be best studied. These maps enable the engineer to measure areas of catchment, and to calculate the discharges from these areas if the proportion of run-off is known.

In locating the dam site it is necessary, first, to have above it the largest possible basin or valley, with a low slope in order to get a maximum storage capacity. Then it should be so chosen that it shall require as short a dam as possible, and yet afford a sufficiently long natural or artificial waste-way to discharge the maximum flood without injury to the dam. Finally, it must be so located that the geological formation shall be favorable. The dam must not be founded on a porous formation, and if of earth, the material of the abutments and foundation must be of suitable quality to afford an impervious connection. If a masonry dam, it must be founded on solid rock. Material for the construction of either class of dam should be convenient to the site. The area to be covered by the artificial lake should contain no porous strata which may lead the water off and waste it. Very careful and detailed surveys will be necessary to ascertain all the above facts, as well as the cost of construction, location of waste way and of discharge gates.

As the general considerations guiding the location and construction of a reservoir for irrigation purposes are very similar to those with which American engineers are familiar in designing storage works for city water supply, only such points as may be of immediate novelty or interest in Indian practice will be described. The modern engineer designs the cross-section of the dam on some theoretic profile which will be the most economical, and all of these cross-sections are very similar to that discussed by Mr. Edward Wegmann, Jr., M. Am. Soc. C. E., in his recent treatise on the "Design and Construction of Masonry Dams." Modified formulas based on Rankine, Delocre, Krantz or Bouvier a regenerally employed. In two of the best and most recent high dams constructed in India, those at Bhatgur and Tansa, the material employed was a hard trap or greenstone, which is abundant there, and the cement is of local manufacture, being made of dirty nodules of limestone found in the ground and called *kunkar*, which, when burned in a kiln, produces a good hydraulic lime nearly equal to the best cement in its resistance to pressure. The Tansa dam is constructed wholly of rough uncoursed rubble, which was merely scabbled on the facing, and was very carefully bedded and laid in the cement. In the Bhatgur dam a concrete hearting was used above where the pressure was sixty pounds to the square inch; below that uncoursed rubble was employed, while the whole was carefully faced with courses of dressed stone. In no Indian dams are stones used larger than three men can carry, as all stones are carried by hand, and in fact all kinds of labor is manual, machinery being rarely employed. The method of quarrying the stone and transporting it by cars to the work under construction is similiar to that employed in America, as is the general construction and charging of the limekilns.

The following are a few extracts from notes made by A. Hill, Esq., Superintending Engineer, on the masonry construction of the Bhatgur dam :

"In the facing the masonry is laid in courses, each of which does not exceed 9 inches in depth, as large stones are too troublesome to handle. The stones are wetted, and mortar and stones are well rammed. Stones for rubble are as large as can be conveniently handled, while the smaller stones are broken up for metal to be used in making concrete. In uncoursed rubble the stones touch along their whole sides, not at points or edges merely, and small chips are used to fill in spaces between large

stones. All awkward projections are knocked off the stones, and they are well rammed into the bed of mortar in which they are laid, which must be at least one inch in thickness. The stones used average 100 pounds weight each.

"Concrete is made of broken metal and river gravel, the former 3½-inch gauge, the latter from ⅛ inch to 3 inches. The proportions are: metal 16 parts, gravel 16, mortar 12. The concrete is laid in two layers, each 4½ inches in thickness, and rammed for twenty minutes and one hour respectively, and is kept constantly wetted for at least two weeks that it may set properly. The concrete is mixed both by hand and machine, the hand mixing being done on a floor with a hoe, the machine mixing in barrels or mixers of metal which are revolved by hand. Metal and gravel are wetted before being mixed. Some briquettes of concrete, 15 x 10 x 10 inches, after setting for six months, ruptured under a crushing weight of from 400 to 800 pounds per square inch.

"In making mortar only sharp, clean river sand is used. The lime is burnt till hydraulic, slaked five hours, stirred and ground for three hours more, and then the clean, wet sand is added in the proportion of one to one and stirred again for two hours more. For testing, this is made into 2-inch cubes or briquettes, which are kept damp and allowed to set for forty-eight hours, after which they are placed in water until tested."

At Bhatgur, the following are average tests:

			Pounds per square inch.
Mortar, 12	months,	crushing weight	815
9	"	"	600
6	"	"	500
3	"	"	300
1 to 2	"	"	250

Some briquettes stood as high a crushing weight as 1 100 pounds per square inch, when one year old. The hydraulic lime used at Vir, which is at the head of the Nira Canal, is perhaps the best in India. Several briquettes of mortar made there mixed in the proportion of 1 to 1, and from eight months to one year old, required a crushing weight of 1 500 pounds per square inch to rupture them. Vir mortar, mixed with 2 of sand to 1 of hydraulic lime, breaks at 800 pounds pressure per square inch. W. Clerke, the chief engineer of the Tansa Reservoir, keeps up a constant and regular series of tests of all mortar used daily

in the construction of that dam, some of the briquettes being allowed to set dry, others under water, and a record of their crushing strength for different ages is kept. The mortar used there gives results even higher than those quoted above for Bhatgur.

The Bhatgur Reservoir is located in Bombay Presidency, about 40 miles south of Poona, at the forks of the Nira River near its head. The topography is very similar to that in Arizona, north of Prescott and near Peach Springs, consisting of high mesas or table lands cut by deep cañons, which open out on broad bottom lands or valleys. At the dam site the fall of the river is 5 feet per mile, and the reservoir backs up stream 15 miles. Its total capacity is 119 600 acre-feet, and its catchment area is 128 square miles, the discharge over which may be as high as $\frac{5}{8}$ of an inch in depth per hour in floods. The maximum flood may be 50 000 second-feet, while in the irrigating season there are scarcely 20 second-feet running in the river at the canal head at Vir.

The dam is 130 feet high above its foundation and 8 feet above high water mark. Its extreme bottom width is 73.7 feet, and its width on top is 12 feet, and is used as a roadway. When full the pressure at the toe is 5.8 tons per square foot or 90.8 pounds per square inch. When empty it is 6.7 tons per square foot at the heel of the dam. The total length of the dam is 4 067 feet, and it curves in an irregular manner across the valley so as to follow an outcrop of rock on which it is founded. The foundations are excavated two feet below the surface of the solid rock and in some places are as deep as 30 feet in order to get homogeneous rock. The greatest flood over the dam may be 50 000 cubic feet per second, and this will be passed off by two waste weirs and twenty under-sluices. These waste weirs are at each end of the dam and are arched over so that the roadway is continuous above. Their total waste-way is 810 feet long, and the flood may pass 8 feet high through them. One consists of forty-five arches of 10 feet span each, the other of thirty-six arches of the same span.

Of the under-sluices fifteen are constructed in the center of the dam, 17 feet apart, and are 4 x 8 feet each, and 60 feet below high water line. With this enormous head they will discharge 20 000 second-feet, which is almost a maximum flood; they have withstood two seasons of flood successfully without any injury to the masonry joints. With full head the velocity through these sluices is 36 feet per second. Above this row of fifteen under-sluices and near the surface of the hillside are two

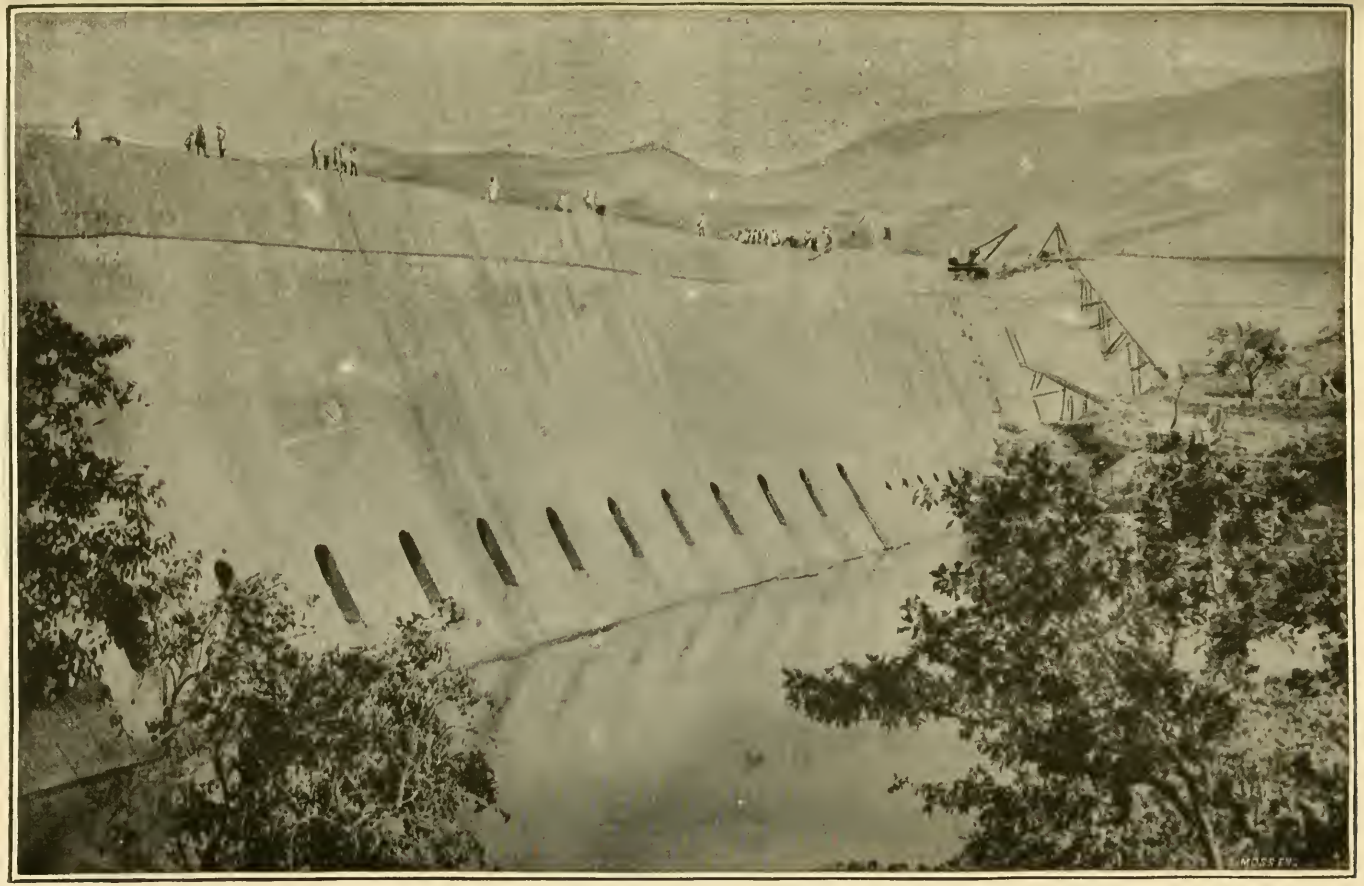


PLATE LXVI. BHATGUR DAM. Vol. XXIII, p. 246.

others of the same dimensions, one 20 feet above the main row, the other 50 feet above them. These under-sluices are constructed with a lining of ashlar masonry with best pointed joints, and they are closed by iron gates which slide vertically. These gates weigh 2 tons each and are protected from injury by floating objects, by a stout grating of wood on the up stream entrance. The gates are raised by a steel screw worked from above by a female capstan screw turned by levers.

There is 20 feet of idle space below the under-sluices and this is expected to silt up. The object of these sluices is primarily to discharge the water of the reservoir, which then flows 20 miles down the Nira River to Vir, where a pick-up weir turns it into the head of the Nira Canal. Fewer sluices, however, would have accomplished this end. The object of making so many was with a view to keeping the reservoir above them free from sediment; this is only accomplished by keeping them open when much sediment is in suspension, and prevents its deposit by drawing it off. It is doubtful if it would clean the reservoir if it once got silted.

A. Hill, Esq., Superintending Engineer, says of this scouring effect: "Scouring sluices have little effect unless the area of the openings is large compared to the area of the floods. To remove silt already deposited they are useless. Those at Bhatgur have a large area compared to the floods, and hence are effective; at Vir their area is much smaller, hence that reservoir has silted even in front of the sluice. If the dam is high and the discharge of the under-sluices will keep the flood level below the full supply level, then they will be efficient. If the dam is low and the sluices will not keep the flood level below full supply level then they will have little effect."

Experience on the Betwa reservoir and elsewhere bears these conclusions out, with the additional statement that their scouring or preventive effect is felt but a very few feet on either side of the sluices and silt will deposit close to their entrance, in other words, they do little more than keep an open channel above them.

Another great dam is that at Tansa, about which there is little that is novel excepting its great size and excellent workmanship and design. The discharge of its catchment area, which is only 52½ square miles, may be 8 000 000 cubic feet per day. The area of the reservoir is 8 square miles, and its available depth, that is the depth above its discharge or outlet sluice, is 20 feet. Its available capacity is 52 670 acre-feet, less

36 800 acre-feet, which is 6 vertical feet lost by evaporation and absorption. Its total capacity is much greater than this. The total length of the dam, which is constructed of the best uncoursed rubble masonry, is 9 350 feet, with an overfall waste weir 1 800 feet long at the south end. The dam is constructed in two tangents, the object being to get the shortest line on a bed-rock foundation.

The object of the great pick-up weir at Vir is to divert the water from the Nira River into the canal head ; incidentally it stores a small amount of water. This weir is peculiarly constructed just above the junction of the Nira and Vir Rivers, the object being to get a sufficiently wide waste way for the enormous flood volume which passes over its entire width. The weir, which is constructed of uncoursed rubble masonry, is 2 340 feet long, 9 feet wide at top, and 43½ feet in greatest height, and is nearly vertical down stream, having a slope of about 4 on 1 up stream.

A short distance below this weir, in a narrow part of the main river, below the forks, is a subsidiary weir 615 feet long on top, 20 feet high, with its crest 24½ feet below that of the main weir. Both are constructed of the best uncoursed rubble with a concrete core. The upper weir impounds 60 000 000 cubic feet and backs the water 12 miles up the river. The greatest flood may be 160 000 second-feet, which would pass 8 feet deep over the main weir, and 22 feet deep over the subsidiary weir. The object of the latter is to give a water cushion on which the floods from the upper weir will fall without jar, and the higher the flood the higher will the water be backed upon the face of the upper weir and the shorter its fall will be.

At the head of the Betwa Canal, near Jhansi, in the Northwest Province, is a great storage reservoir, the dam of which, like that at Vir, is an overfall weir in its entire length, and water is diverted by a canal heading above the weir. During the irrigating period the river runs quite dry, while in great flood it may discharge 750 000 second-feet. The dam is constructed of rubble masonry and is 4 300 feet long, situated in the widest part of the river in order to give sufficient waste-way for the great floods which may pass 17 feet deep over the weir crest. Its extreme height is 60 feet, and the height of overfall is 21½ feet to a water cushion formed by a subsidiary masonry weir 10 feet in extreme height.

The cross-section of this weir is peculiar, and appears to be unnecessarily heavy; it is 15 feet in width on top, nearly vertical on the down

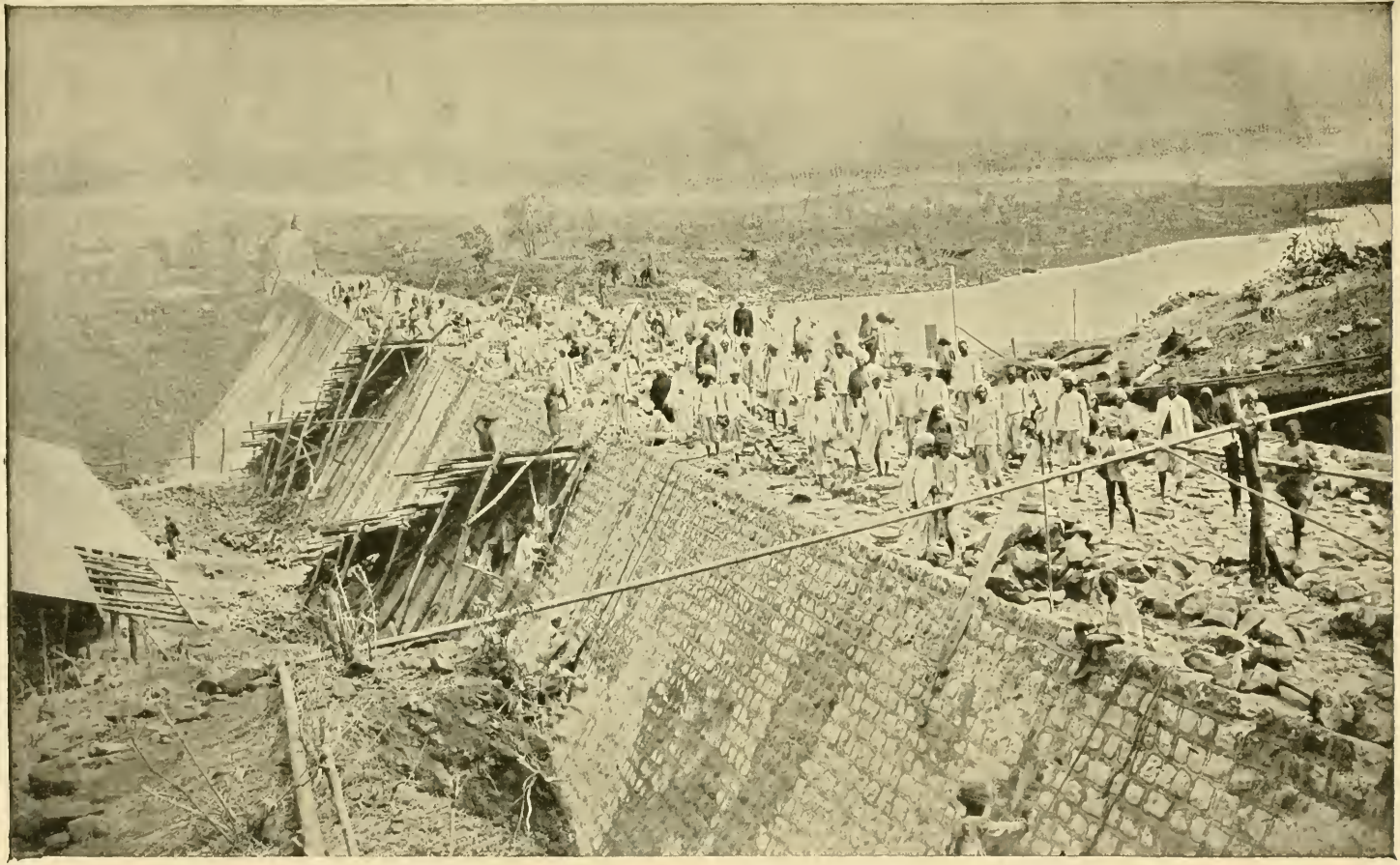
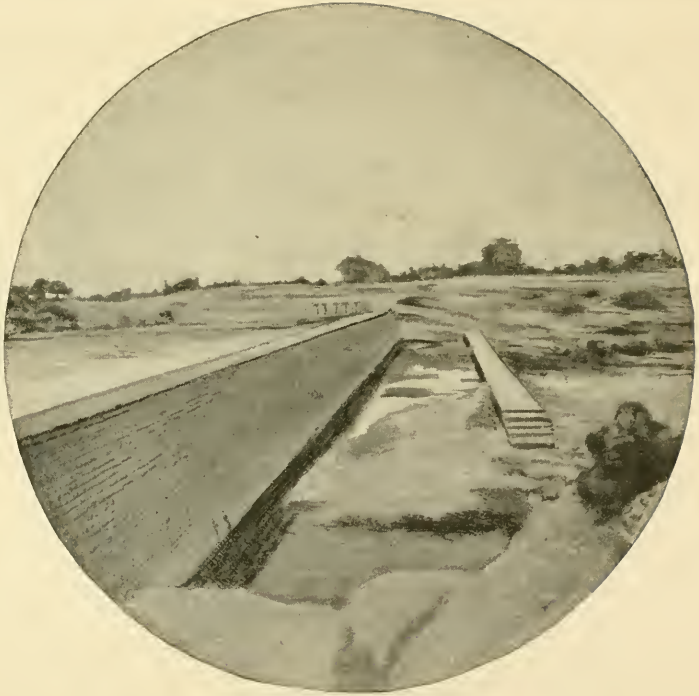


PLATE LXVII. TANSA DAM. Vol. XXIII, p. 248.

GENERAL VIEW OF FOUNDATIONS.

CARRYING STONE FOR CONSTRUCTION.

DROPPING AUTOMATIC SLUICE GATES,
SOANE CANAL, DEHREE.



MAIN AND SUBSIDIARY WEIR AT VIR,
NIRA CANAL.

stream face, which terminates near the bottom in a square masonry apron, and has a curving slope up stream. The great top width is intended to enable it to withstand the jar of the overfall; the same is the reason for the heavy masonry toe, while its vertical face down stream is intended to allow the water a clear overfall to the water cushion.

The available net storage capacity, that is above the canal bed, is 36 800 acre-feet, though the executive engineer, Mr. W. P. Von der Horst, says that the lower 6 feet are of little service, as the head above is not sufficient to make the water run out freely.

The canal heads just on the upper side of the dam, in which latter are four under-sluices intended to scour out silt and prevent its deposit at the canal head. These gates are worked by screws and capstans from above, and have an additional set of drop-gates, to be used to ease the pressure on the main gates. The openings are at the bottom of the dam and are 16½ x 6⅔ feet, with a masonry "Vena Contractor" at their entrance. The regulating gates at the canal head are five in number, and have each two openings one above the other, the lower being under an extreme head of 25 feet, and the upper of 12 feet, the object being to admit water to the canal from the upper one when the reservoir is sufficiently full, and through the lower openings when the water is low, thus avoiding the necessity of working the gates under the full head of pressure.

Tanks.

Tanks closed by earthen dams, or as they are known in India, *bunds*, were constructed in great numbers by the natives many centuries ago, and in some places, notably in the Deccan, Madras and Mysore, in Southern India, and in the Central Provinces, there are immense numbers of them. These old tanks were usually rude in construction, little care having been taken to economize in labor, and as a consequence the dams are usually much wider than there is any necessity for.

Recently the British engineers have constructed several very large tanks, though most of the higher ones have masonry cores or are paved heavily on the up stream face.

The conditions controlling the location of a tank are similar to those for a reservoir; the water supply must be adequate to fill the tank, the location must be such that a suitable dam can be constructed at reasonable cost, and a waste-way must be provided. In the location of the waste-way for a tank more care must be exercised even than for that of

a reservoir, for if the water is even permitted to rise too high on the dam, much less to discharge over it, it will speedily cause its destruction, as it is constructed of earth. It is desirable always to allow waste water to flow off by a side channel, which may be closed by a low rock or crib work weir, or may even consist only of an open rock cut. By such means there is no danger to the dam by having the shock of water falling over it. It may, however, be necessary to have a masonry waste-way in the dam itself, but this is not advisable. Such a waste-way may be made by constructing a portion of the dam of masonry, the top of which must be a few feet lower than that of the main dam.

It is possible when the catchment of the tank is small, to safely provide for wasting flood waters through an outlet or discharge channel in the dam. Such a discharge channel is necessary in order to draw off the water for irrigation, and must be of sufficient capacity to empty the tank in the shortest irrigating season. The level of these outlet sluices is generally that of the bed of the tank, but there may be one or more higher outlets to irrigate lands which are above that level. These sluices are expensive works, and require a well constructed masonry or iron conduit to be carried through the dam, emptying into the canal at one end and receiving its water supply at the other end through a well or water tower from which its admission can be controlled. It is usual to make the outlet tunnel of good masonry, and it must not be smaller than 2½ feet high by 2 feet broad, in order to permit a man to go through to examine and clean or repair it if required.

One cause of the breaking of Indian tanks is that great numbers of them are often formed in one valley, the bed of one beginning where the cultivation from that of the next above it ends, so that the breaching of one of these often results in the destruction of a succession of those below it. According to Captain Green there are over 20 000 tanks of all sizes in Mysore State alone. These usually have from one to four masonry escape outlets each.

Large deposits of silt occur, which, in the course of time, destroy the usefulness of the tank. At the beginning of the rainy season this is largely removed by having the mud stirred up by hand, and permitting the flood waters to carry it off through the discharge sluices. This, however, is only partially effective like the under-sluices in a reservoir. It is best to gradually increase the height of the dam as the tank fills up, though there eventually comes a limit to this increase of height.

According to Col. Baird Smith, the Chumbrumbaukum tank in Madras has an area of 9½ square miles. Its capacity is 69 000 acre-feet. Its dam is over 3¼ miles in length, ranging from 9 to 28 feet in thickness and from 16 to 28 feet in height, and is constructed of earth. It has six waste weirs, having a total length of 676 feet.

The embankment of the Kabra tank is 620 feet long, and is built of stone set with limestone mortar, backed on the lower side by an earth bank as high as the storage wall, which latter is vertical on its up stream face. The greatest height of the dam is 33 feet, and the contents of the tank is 132 800 acre-feet.

The Ekruk tank, in Bombay Presidency, is closed by a dam 7 200 feet in length, the masonry portions being 2 730 feet long. The maximum height of the earth bank over the stream is 72 feet, the water face being pitched with stones two feet in length. The area of the tank is 6½ square miles, and its storage capacity is 41 700 acre-feet of water, the maximum depth of which is 60 feet. The waste weir is at the northern end, and consists of a channel 250 feet wide, carried through a spur and leading into a side drainage course.

One of the greatest tanks, which is formed by a dam constructed simply of earth, is the Ashti tank in the Sholapur district in the Deccan. This great work was undertaken as a famine relief measure in 1876, under the direction of C. T. Burke, C.E. The project comprised a tank on the Ashti River, formed by an earthen dam 12 709 feet long, 58 feet in maximum height, covering 4.2 square miles, and having a storage capacity of 34 500 acre-feet of water. Two canals, one on either side of the Ashti, extend as far as its junction with the Bhima River. These canals command 25 270 acres of irrigable land. The top breadth of the dam is 6 feet, the width at full supply level 42 feet, and the height of dam above full supply level 12 feet. Above this point the slopes are 1½ to 1, and below it the inner is 3 to 1, and the outer slope 2 to 1. The sand and silt were cleaned off the surface along the dam site, which thus rests throughout on a firm foundation. There is no puddle wall, but a puddle trench, the filling of which rises 1 foot above the ground surface, extends throughout the whole length of the dam, and is 10 feet in width, and has been excavated down to a compact and impervious bed. It consists of two parts sand and three parts of black soil. The central third of the dam is built up of selected material of black soil, extending in a triangular section from the dam crest, where it is 6 feet

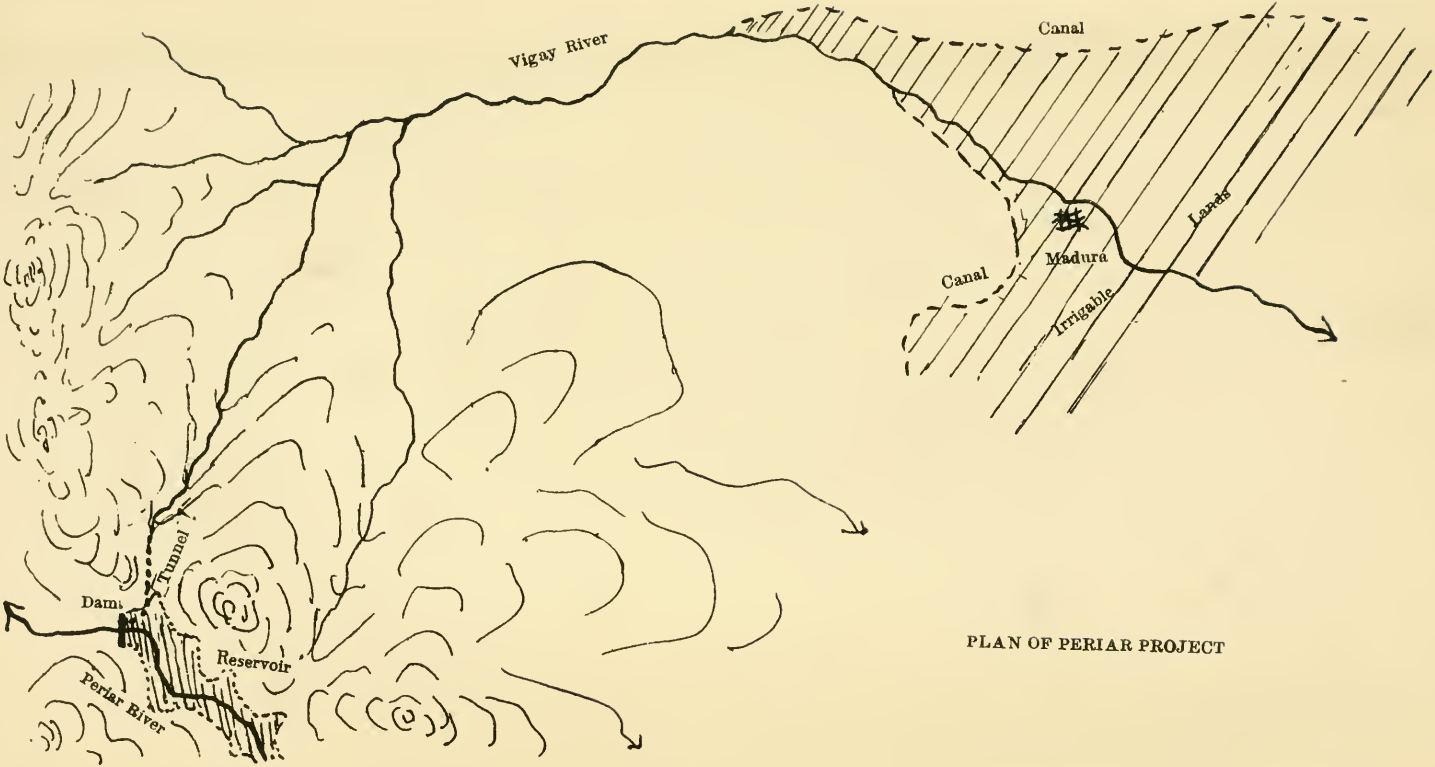
wide, to the base, where it is 60 feet in width. Outside of this are two triangular sections, composed of the brown soil of the country, and this is faced outside with from 1 to 15 feet of puddled sand and black soil, while a similar puddle on the inside is paved with 6 inches of large stone pitching to enable it to resist wave action. The waste weir discharges around a saddle into a channel separate from that on which the dam is built, and is excavated in rock, being 600 feet wide, 800 feet long, with a slope of 1 in 100. Its discharging capacity is 48 000 second-feet, which will cause water to stand 7 feet high over its sill; the latter is 12 feet below the crest of the dam.

Periar Project.

Probably the most interesting combination of storage work and irrigation canal, and one that has been sanctioned as a productive work, is that for the diversion of the waters of the Periar River for the purpose of irrigating the Viga Valley in Madras. According to the report of Major J. Pennycuick, R. E., the object is: first, to construct a dam to close the Periar Valley which will store 306 000 acre-feet of water, of which 156 700 acre-feet are available for irrigation; second, to construct a tunnel with sluices and regulating gates through the watershed dividing the Periar from the Vigay Valley, for the purpose of drawing off this storage; thirdly, the construction of the works for controlling the supply from the Periar while passing down the valley of the tributary by which it reaches the Vigay; fourth, the work necessary for the regulation and distribution of this supply for the irrigation of 140 000 acres of land in the Vigay Valley.

The dam is to be constructed throughout of concrete and is to be 155 feet high above the river bed, exclusive of a parapet 5 feet high. It will be 1 230 feet long and 12 feet thick at top, and 154 feet thick at bottom, with a cross-section similar to that obtained by M. Bouvier's formula. There will be two escapes around saddles at either end of the dam; one will be cut down through the solid rock to the requisite level, and will be 420 feet long; the other will be partly cut down, and partly constructed by raising a low masonry weir 403 feet long, giving a total length of escape at this end of 500 feet. At a distance of 60 feet from this escape weir will be a low subsidiary weir 10 feet in length, with its crest 30 feet below the escape weir to form a water cushion.

In a side valley below the main dam will be made a cutting towards



PLAN OF PERIAR PROJECT

PLATE LXX.

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the Vigay watershed, 21 feet broad, with a fall of 1 in 440. At 5 400 feet this will be 30 feet deep in rock, and then will commence the tunnel, with an area of 80 square feet and a fall of 1 in 75. At its lower end the tunnel will connect with a cutting similar to that at its upper end, through which the water will pass into a tributary of the Vigay, down which it will take its way to that river. This cutting will be 160 feet in length, the tunnel being 6 650 feet long.

The reason given by Major Pennycuick for the adoption of concrete in the construction of the dam, is that it is really uncoursed rubble reduced to its simplest form, the materials used in either case being identical, as is the resistance of the two to percolation and crushing. The selection between the two depends entirely on their relative cost, which will here be high for rubble, because the cost of labor is abnormally expensive, while the facilities for using labor-saving machinery in mixing concrete are unusually great.

Beyond the general plan as above outlined there is nothing especially worthy of note in the regulating gates or distributive works. There are some interesting devices to be used in controlling the water of the Periar during the construction of the dam, and some of the diversion weirs in the Vigay Valley are worthy of note did space permit.

DISCUSSION.

J. B. Johnson, M. Am. Soc. C. E.—What is the maximum velocity in the canal, in earth or clay ?

Herbert M. Wilson, M. Am. Soc. C. E.—It differs largely according to the character of the soil. In light sand the mean velocity ranges on Indian works from 1¾ to 2½ feet per second; in a more clayey soil, from 2 to 3 or even 4 feet. 3 feet per second might be taken as a good average ; in gravel the velocity may be far greater. In stiff clay, at the 95th mile on the Ganges Canal, a velocity of 4.1 feet produces no erosion.

James B. Francis, Past President Am. Soc. C. E.—The flow of the Mississippi at New Orleans during the recent flood, it is said, was 1 300 000 cubic feet per second ; that is no larger than some of these quantites.

Desmond Fitzgerald, M. Am. Soc. C. E.—I think I voice the sentiment of every hydraulic engineer here when I say that I have listened to Mr. Wilson's paper with utmost enjoyment. It is refreshing

to listen to these large figures when we have been dealing perhaps with a few cubic feet per second. It does not seem to me, however, that our practice of providing heavy aprons for falling water, which has been founded on many failures, is likely to be upset by some things that we have heard about water cushions. Even with a water cushion the effect of scour is very great, and that must be provided for. It is difficult for me to suppose that I can carry water over a 50-foot dam by simply having a water cushion.

Mr. Wilson.—Several of the photographs show weirs. At Vir are two, the main one 40 feet high, and a subsidiary weir 20 feet high, over which flows in the first case as much as 90 000 second-feet of water. Over the crest of the higher weir I think the depth of the water in great flood was 8 feet, and the depth over the lower weir was 13 feet. In the case of these weirs the foundation is rock and the surface soil chiefly of hard pan, the abutments on both sides striking the rock. There is an island just below the main weir which the engineers desired to wash away; it backed the water up against the weir, and caused it to force a passage around the island. The material of this island is 5 or 10 feet of gravel on top, underlaid by a hard pan in which limestone nodules are found. Channels were cut through the island (and the same thing was attempted at the Betwa weir), and streams were directed into the channels, and in the course of three or four years these islands have scarcely eroded it at all.

Mr. Fitzgerald.—What does the water fall directly upon, hard pan?

Mr. Wilson.—The water falls over the weir and on the rock bottom, churning against the island and the toe of the weir, and the channel has been excavated about 15 feet wide between the weir-toe and the island so as to give a sufficient passage around both sides of the island. The weir was carried all through the island gravel and founded on bed-rock, as is the case with the Betwa.

Mr. Francis.—We have a natural water cushion at Niagara. It has made a water cushion for itself, the depth of water being very near the height of the fall itself. The bottom rock is said to be not very hard.

Mr. Franz A. Velschow.—So far as I have been able to follow Mr. Wilson I have found his able description of the Indian irrigation works correct and conforming to the English publications on the subject. But I understood Mr. Wilson to say that, judging from the general configuration of the two countries, and the direction of the flow of their principal rivers, he considered the climatic conditions of India and the Western States as somewhat similar, particularly with regard to the conditions favorable for irrigation, and, as I have published elsewhere,

there appears to me to be a vast difference in this respect. India is a country comparatively easy to irrigate, as it possesses the most important and almost necessary condition for all efficient irrigations—an ample and never failing supply of water in the principal rivers. On the other hand, in a country like Australia, first, it sometimes fails to rain for several consecutive years, and one may then walk dry-shod across the largest river, the Murray-Darling, which, at other times, may fairly compare to the Amazon River in magnitude. Here the only remedy against drought is extensive storage-works, which are very seldom needed in India.

It appears to me that the conditions in the Western States lie somewhere between the two extreme cases mentioned here, and consequently much storage will have to be resorted to in these Western States to make irrigation successful there. As to the immediate cause of these dissimilarities of conditions I beg to refer to my paper on the cause of trade-winds.*

Mr. Wilson.—I am glad this question has been brought up. In India, in the regions which are mostly irrigated, take the northwest province as an example, the rainfall varies from 20 to 40 inches per annum, is scarcely ever as low as 20 inches, and is, in one place, on the lowlands of that province, as high as 60 inches. Most of this rainfall occurs during the summer months of the year, the monsoon period, when crops are raised without the aid of irrigation. Irrigation is practiced during the autumn or dry season, during which from 6 to 8 inches only fall in the north west provinces; 8 inches would be high. In the Sinde it is less than 2 inches. The autumn crop is the one that I discuss chiefly in this paper. This crop is raised under climatic conditions similar to those of our arid regions. The chief object of water storage in India is in order that the autumn crop may be grown. The population has become so dense that the summer or wet weather crop alone is not sufficient to supply the needed food, and by storing the water they are enabled to raise the second or autumn crops.

The studies made by the hydrographic engineers of the United States Geological Survey show that the annual precipitation west of the Powell or 100th meridian is below 20 inches. In Southern California in the neighborhood of Yuma, it is less than 4 inches; in Colorado, it is somewhere near 15 inches; in Montana 13 inches is the average precipitation. The climatic conditions, and the amount of pecipitation during the irrigation season in the arid west, are almost identical with those of the autumn season in India. In Montana the precipitation during cropping season varies from 4 inches to about 7 inches, while the annual precipitation is as high as 13 inches.

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* See *Transactions*, Vol. XXIII, August, 1890.

H. C. Meyer, F. Am. Soc. C. E.—What about getting a revenue for water? The failure, I understand, was due to the fact that the water was not taken in the dry season. Is there any provision for getting a revenue every season?

Mr. Wilson.—No; they cannot compel the people to use the water. The land is the property of the British Government, and the revenue is collected on the sale of timber from the forests, and particularly on the lands under cultivation. The revenue collection is under a separate force, called the Revenue Service of the Government, and they assess the taxes on the land according to the crop raised, as given them by the engineers in charge of the works. Nearly all works and all land are the property of the Government. The water is applied for by communities. At the end of the season, before the crop is harvested, the collectors examine the crops and assess the land revenue according to the crops cultivated. The reason for this is that such crops as sugar-cane and rice use a great deal more water than wheat, etc., and are hence assessed higher. If the irrigators do not ask for the water the Government does not charge for it. The water which is supplied from wells is taxed just the same as from canals.

Mr. F. H. Newell, Hydrographer, U. S. Geological Survey.—One of the most striking facts brought out by Mr. Wilson's paper, beyond the immensity of the Indian enterprises, is the care and thoroughness with which each detail is elaborated, and the wonderful permanence of all structures.

For each irrigation scheme careful surveys are made, the hydrography of the country studied, and estimates prepared. These in turn are examined and criticised from all standpoints—financial, engineering and sanitary, and finally construction is begun in a most thorough and substantial manner. Here, on the other hand, irrigation has grown up in a haphazard way, with little or no preliminary examination, and the schemes are pushed with great energy, having more regard for quick returns than for future permanence of works or water supply.

Of all structures made by man those having to do with irrigation are by far the most permanent. In the older countries where civilization was born—for the home of all ancient civilization was in arid lands—the canals still remain in use, although in many cases neither history nor tradition tells of their builders.

The history of other nations has shown that an irrigation canal or system of ditches once developed—whether located poorly or not—is rarely improved, and if at all, with the greatest difficulty and expense. All rights, individual, corporate, or municipal, quickly become clustered around that system of water distribution, and he who would better it must literally buy the whole country before it can be changed, for in arid regions running water is the one thing which gives value to

land. From this fact is seen the great importance of correct location, proportioning and designing all parts, and the necessity of the utmost skill and forethought in projecting the canal systems for all time to come.

Since the evils of poor engineering are so far-reaching, it is pertinent to inquire what is the actual situation in the States and Territories of the arid region? how have the water systems been laid out? and what is their present condition?

Take, for example, Utah. This Territory, from its central position geographically, is a fair representative of the arid region. Irrigation was begun here by Americans first, and has been developed entirely uninfluenced by native, Pueblo Indian, or Mexican customs.

The original settlers were northern men, natives of humid regions, to whom an irrigating ditch was a curiosity. Led on by religious zeal, and driven into the desert by fanaticism, these men settled in what was at that time a desolate spot, and from the first were compelled to learn the art of irrigation by trials and mistakes. With no tools but farming implements, and the engineering skill of the carpenter or blacksmith, the canal systems of this Territory were laid out, and have increased until all the available water has been appropriated.

As a consequence of this manner of construction the ditches are of the simplest character. The head works are usually of rough boulder and brush-wing dams, swept away by high water and rebuilt after each flood. No attempt is made to raise the water by weirs. The alignment of the ditches is astonishing, a ditch stopping little short of crossing itself in its windings, following contours without regard to length or loss of grade. The grade itself varies from point to point, and the cross section is constantly changing.

Provision for carrying off or diverting surface drainage is rarely made. When a local storm occurs it merely washes out a portion of the ditch, the water is shut off, and all the farmers join at once in repairing the breach.

Main irrigating ditches are frequently duplicated many times. This usually arose in the following manner: the first settlers being of course poor and in urgent need of a food crop, joined in taking out a ditch to water the most convenient lands, not always the best. The next comers, with perhaps some of the first owners who were not able to get all the water they needed, located a new ditch a little higher and commanding more land, starting a few hundred rods further up the stream and running on the edge of the river bank a few feet above the old ditch.

In turn a third, fourth, and even fifth ditch was taken out, each one higher and larger—and usually a little better built as the community became richer. The last comers, with perhaps the best canal, have only the surplus or flood waters, while the others own the right to the water in the order in which the ditches were built—the first ditch taking all

the water which it first appropriated, the second ditch its share next, and the third in turn, and so on.

Such a system is at the same time the cheapest and most wasteful. The first cost is at a minimum—little or no capital is required, and only the simplest form of association for common benefit. The annual tax is also exceedingly low, almost insignificant. Actual figures of both these items are very difficult to obtain, as the class of men who built these ditches are not accustomed to estimates. Careful and widespread statistical inquiries have, however, enabled the following approximations to be made: with wages for a ten-hour day at $2 for a man, and $3.50 for man and team, using plow or scraper, the first cost of an irrigating system is from $1 to $15 per acre, with about $6 per acre as average. The annual tax for maintaining the canal and all supervisions is from twenty-five cents to $1 per acre, averaging about fifty cents. This does not include extraordinary washouts and similar calamities when all hands join in repairs.

Although the cost is so small, yet from another standpoint the loss is enormous—both in wastage of water, which means less land to be cultivated, and loss of crops through accident. The duty of water is very low, 66 acres to the second-foot for the water utilized, is an average of a large number of independent estimates. For the water entering the headgates the duty is much lower. At the average rate of fifty cents per acre this would only amount to a water duty of $33 per second-foot, against $113 as cited by Mr. Wilson for India. However, there are other drawbacks which render this apparently cheap water not so inexpensive. The unstable character of important portions of the canals, as headworks and flumes, the unregulated liberty of appropriation of river water, with the consequent quarrels over water division, make the real cost far greater than it appears.

The American irrigating canals compare more nearly in size with the distributaries of India, and thus the remarks by Mr. Wilson under this head have direct bearing. He shows that the English engineers have demonstrated by detailed experiments the greater relative cost and loss of water in the smaller distributaries. This fact concerning the great waste of water by the parallel systems of small ditches cannot be too greatly emphasized, when we see to what extent this is being done in our country, where water is already scarce.

While the actual figures obtained in India concerning this relative loss do not hold good here, yet we have every reason to believe that the waste is as great, and probably greater, since our small ditches are more poorly made than the large ones.

On another point—the keeping of the canal surface above the level of the country, and condemnation of stops or dams in the ditches, the opinion of the Indian engineers deserves heed.

The essential difference between irrigation in India and in the United

States is, that while there the canals are owned and built by the government, and regulated not for profit alone, but directly for the benefit of the people and protection from famine, in the United States irrigation has sprung from private enterprise, and the government has as yet, in legislation, hardly recognized its existence. Originally the farmers built the irrigating ditches solely as a necessary adjunct to cultivating the soil. Of late years, however, the great increase in value of the land under ditch has attracted the attention of capitalists, and larger and less wasteful canals are rapidly being built.

Water appropriation has gone on unchecked and unregulated, apparently under the motto of each man for himself; the quantity of water carried by many streams being claimed many times over. Quarrels about water divisions arise not only between rival individual appropriators, but between counties, districts, States and Territories, and even involve the United States and Mexico.

In how far the State or nation is justified in pursuing this "*laissez faire*" course must be left to legislators. All engineers concede, however, that great advancement will be made when the various States and Territories adopt regulations and inspection suited to local needs, and the United States broad and comprehensive legislation on the subject. On this point one of our best engineers, Elwood Mead, Territorial Engineer of Wyoming, says in his last report: "These evils [defective works and wrong locations] will in time undoubtedly disappear, but they could almost wholly have been obviated by the exercise on the part of the Territory of an intelligent preliminary supervision over the location and construction of all irrigation works."

The plates illustrating the paper by Mr. Wilson on Irrigation in India are :

Plate LXII, Nadrai Aqueduct.
" LXIII, Solano Aqueduct.
" LXIV, Drainage Map showing arrangement of Distributaries.
" LXV, Ganges Canal—Ranipur Superpassage.
" LXVI, Bhatgur Dam.
" LXVII–LXVIII, Tansa Dam.
" LXIX, Plan of Periar Project.
" LXX,
" LXXI, Map of India.
" LXXII, Cross Sections of Weirs, Ganges, Soane, Agra Nira and Betwa Canals.
" LXXIII, Cross Sections of Bunds; Syphon Superpassage; Kao-Nulla Syphon Aqueduct; Double Gates, Flushing Sluices, Soane Canal.

PLATE LXXI.
TRANS. AM. SOC. CIV ENGRS.
VOL XXIII No 454
WILSON ON
IRRIGATION IN INDIA

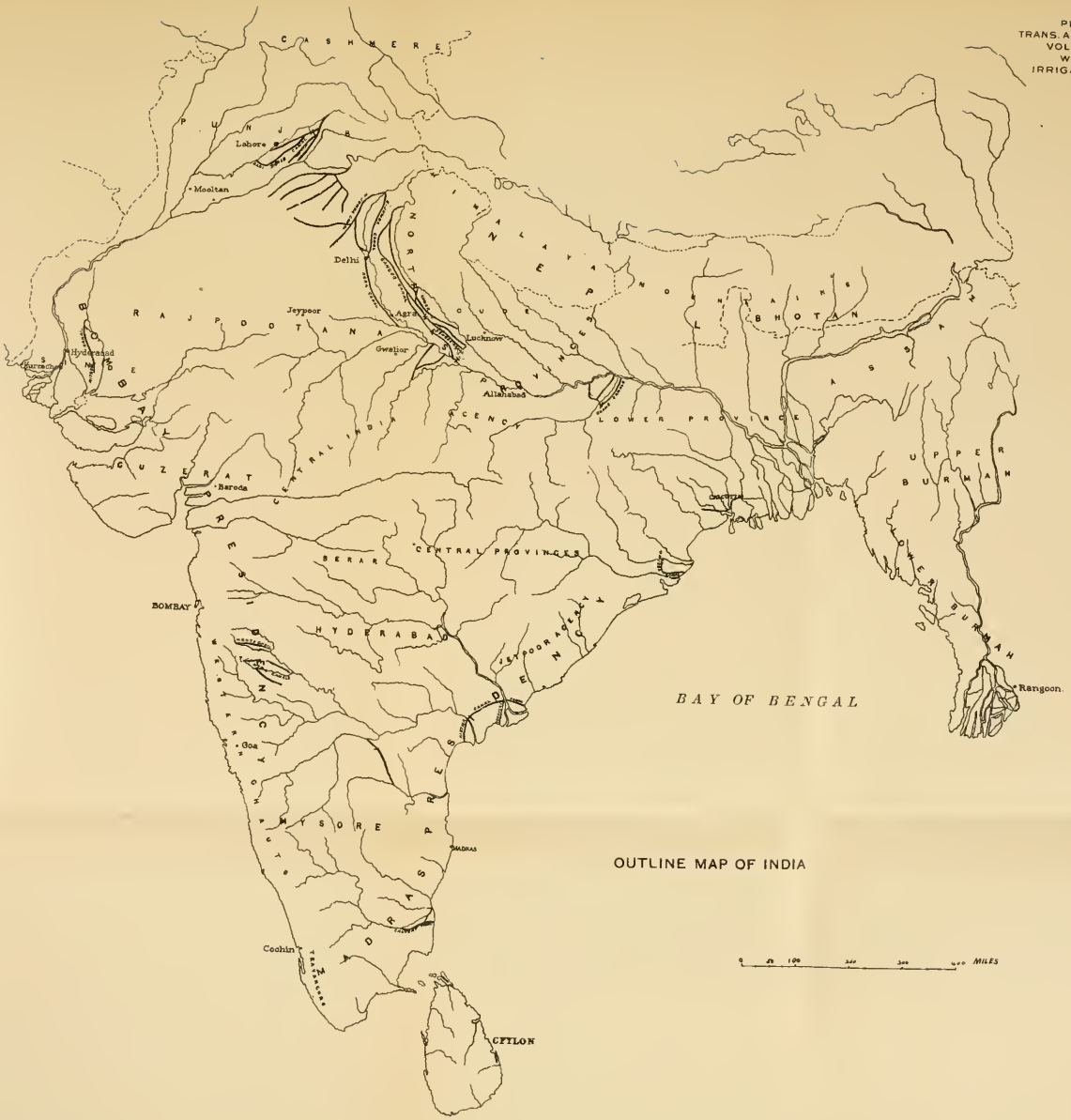
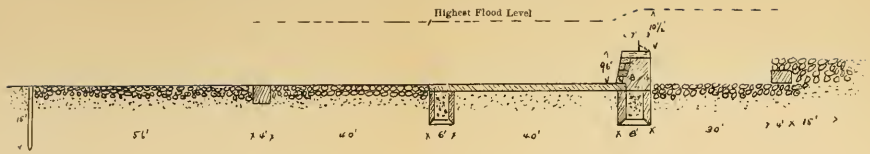
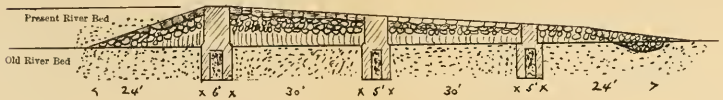


PLATE LXXII
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII. No. 454
WILSON ON
IRRIGATION IN INDIA

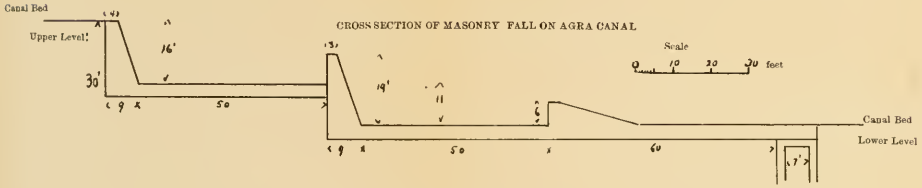
CROSS SECTION OF WEIR AT NARORA, LOWER GANGES CANAL



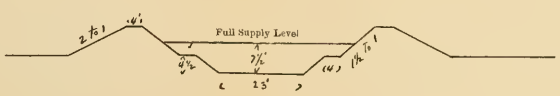
CROSS SECTION OF WEIR AT DEHREE
SOANE CANAL



CROSS SECTION OF MASONRY FALL ON AGRA CANAL



CROSS SECTION OF NIRA CANAL



CROSS SECTION BETWA WEIR

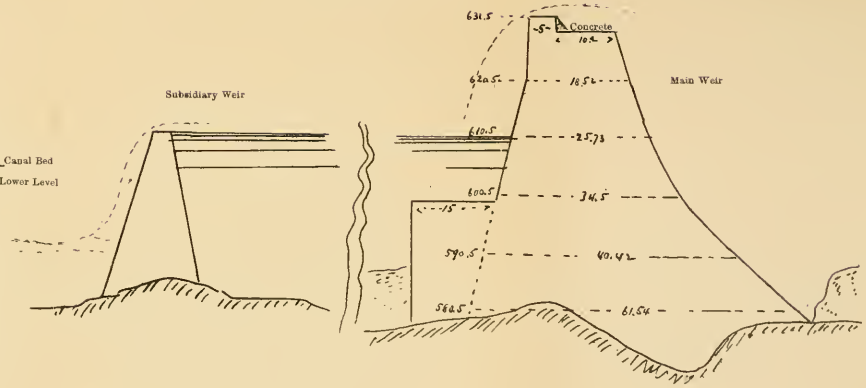
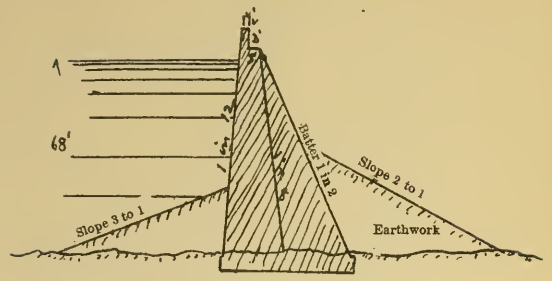
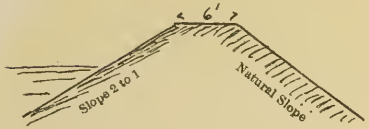
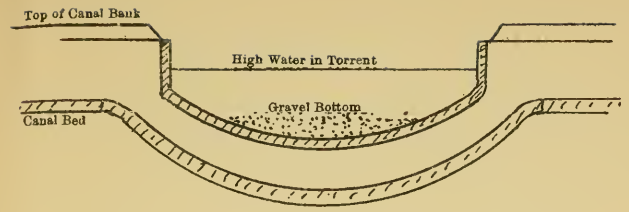


PLATE LXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII Nº 454.
WILSON ON
IRRIGATION IN INDIA.

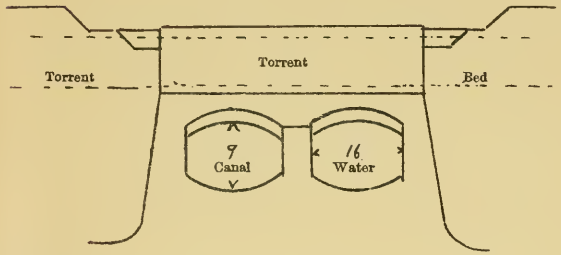
CROSS SECTIONS OF BUNDS

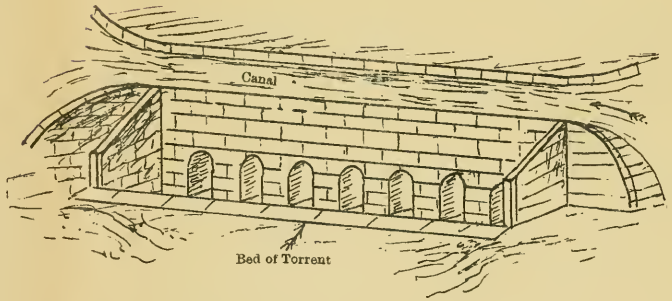




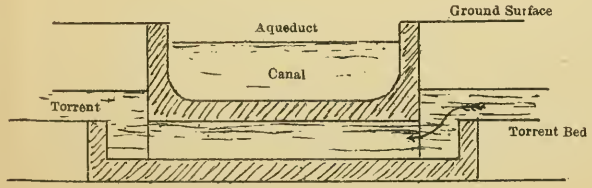


SYPHON—SUPERPASSAGE

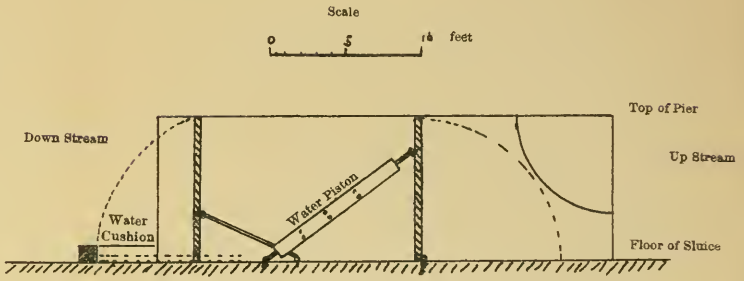


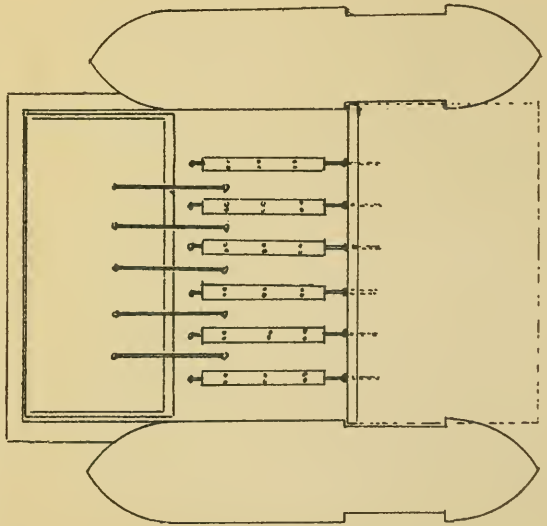


KAO NULLA SYPHON-AQUEDUCT



DOUBLE GATES, FLUSHING SLUICES, SOANE CANAL





AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXIII.—November, 1890.)

PINE STRINGERS AND FLOOR-BEAMS FOR BRIDGES.

By ONWARD BATES, M. Am. Soc. C. E.

WITH DISCUSSION.

Early last year the writer announced his intention of presenting at the Annual Convention of the Society a paper on timber trestle bridges, and failing to carry out that intention, he promised the paper should be ready for the Convention of 1890.

An uninterrupted pressure of professional work has interfered with the fulfillment of his promise, and to redeem it in part he offers some tables of tests of the transverse strength of pine timber, with such comments as he is able to make from a study of the subject.

It has seemed to the writer that the subject of wooden stringers for bridges has been neglected by our profession. Upon most of our western and southern railroads the majority of the bridges are of wood, and the track is carried by longitudinal stringers or transverse floor-beams,

subjected to bending strains. The railway company which it is the fortune of the writer to serve has in use over 100 miles of such bridges, and the problem of economical maintenance consistent with safety, involves large expenditures annually. This being only one road of a great many, whose practice is similar, the writer feels that his partial presentation of the subject will interest many of our members, and he trusts to be rewarded with the results of their experience in the discussion of his paper.

The present practice of great axle loads and close spacing of wheels of rolling stock brings heavy concentrated loads on bridge stringers. An investigation of the weights of locomotives, snow-plows, etc., led at once to the question of the strength of wooden stringers. Literature on this subject was found to be scarce, and with the exception of the series of valuable tests of wooden beams, made by Professor Lanza, the writer was unable to find any which would apply to his practice.

The tests which are represented in the tables accompanying this paper were made to determine the strength of pine stringers and floor-beams, as are commonly used in bridges. Tests are numbered in the tables consecutively, and in discussion of them will be referred to by number. Nos. 1 to 40 inclusive were made by Mr. George Gibbs, Mechanical Engineer, on an Olsen testing machine. Nos. 41 to 67 inclusive were made by Mr. A. J. Hart, district carpenter, with a machine constructed for the purpose. All computations in the tables were made by Mr. W. H. Finley, Assistant Engineer.

Photographs are presented to the Society, numbered to correspond with the numbers in the tables, which show clearly how the tests were made. Plates LXXIV and LXXV are reproductions of two of these photographs. The method of obtaining deflections, which is not shown in the photographs, is as follows:

A nail was driven into the side of the specimen at each end and at the middle of its height. A fine wire was fastened to the nail at one end and passed over the nail at the opposite end with a weight suspended to it. This weight took up any slack which would be caused in the wire by the shortening of the specimen in bending, and kept it taut and straight. The deflections were measured with a steel scale at the middle of the stick, and a number of deflections were taken for each test, in order to reduce errors by averaging the moduli of elasticity. The measurements of deflection and calculation of the moduli of elasticity were

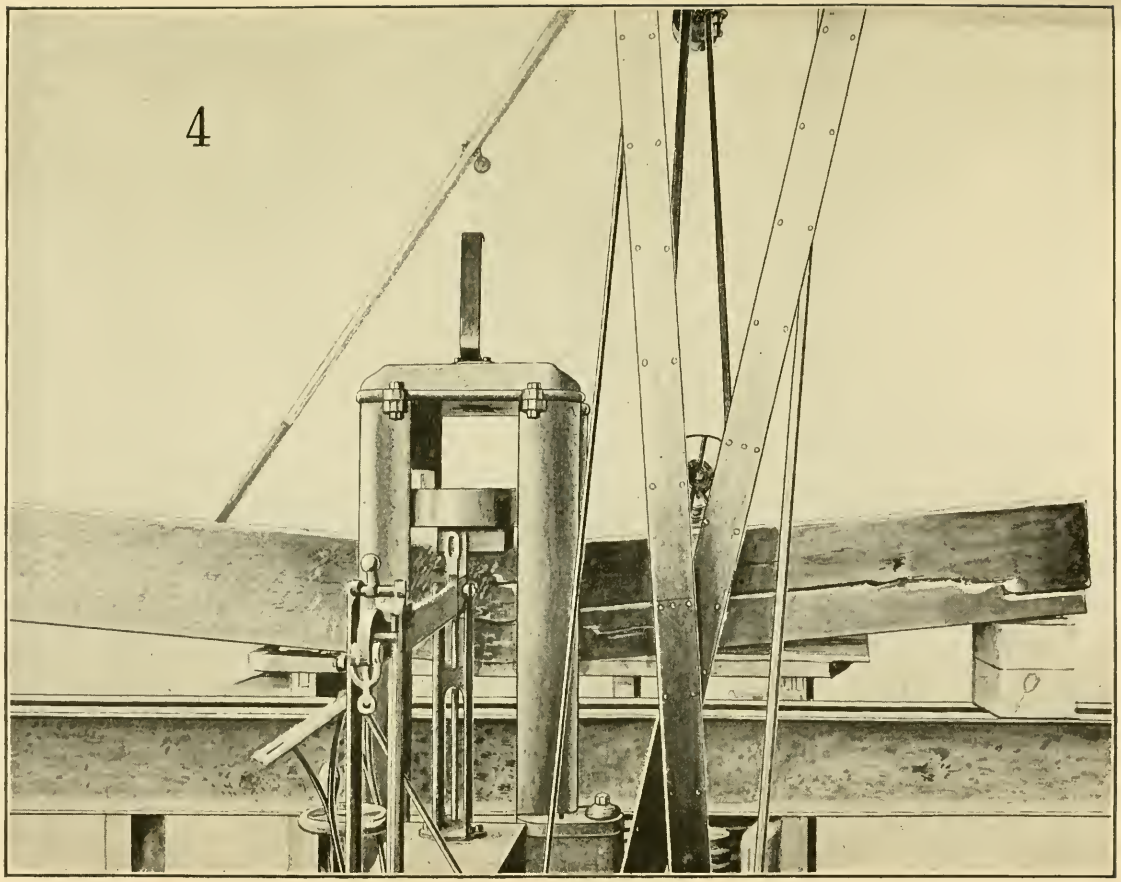


Plate LXXIV. Vol. XXIII, p. 262.

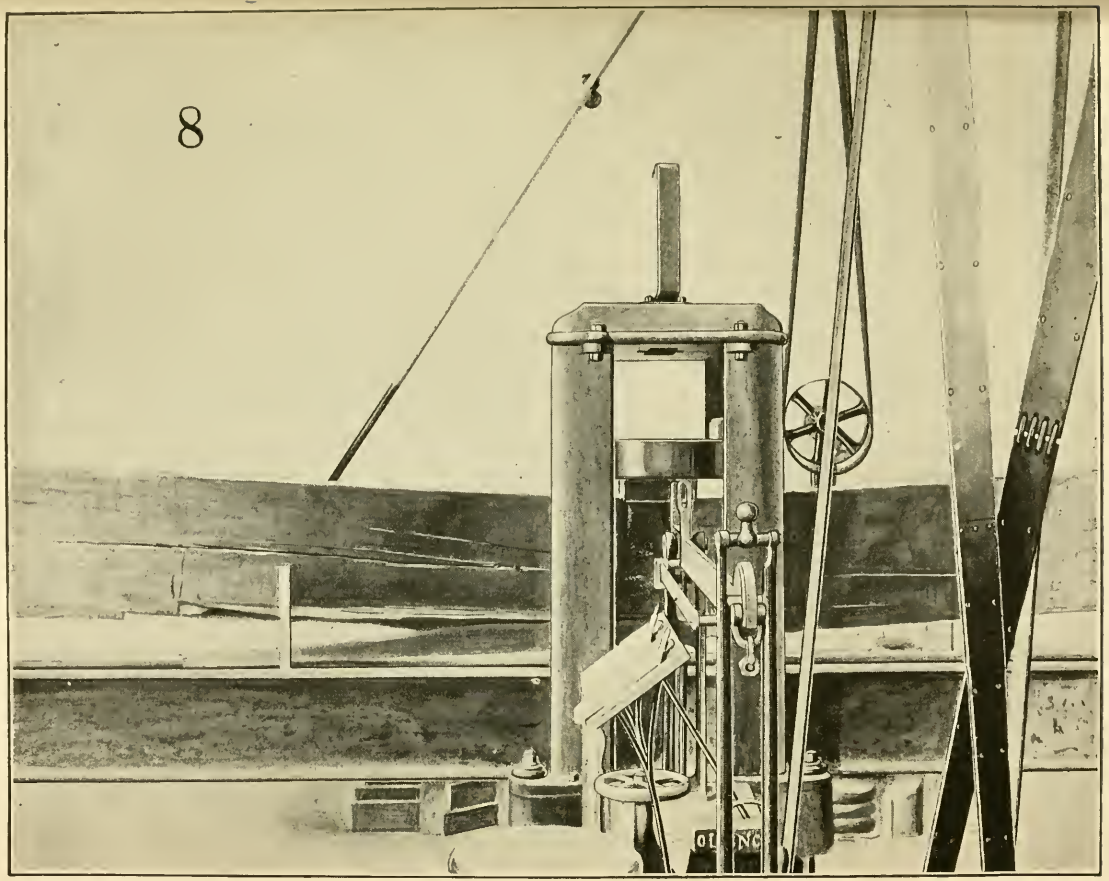


Plate LXXV. Vol. XXIII, p. 262

tedious, and only attempted on the first forty of the tests. As these moduli are only useful in calculating the deflections of beams under given loads, the same importance was not attached to them as to the moduli of rupture. The general average of the forty tests being 1123090, it would seem that the common practice of assuming a modulus of 1200000 cannot be far from being correct for white or Norway pine.

The moduli of rupture given in the tables are lower than is to be found in any of the text books to which the writer has access, but he believes the results of his tests are nearer what may occur in actual use than those of any series of tests made on smaller specimens. He quotes Professor Lanza as the best authority on this subject, and on page 537 of Lanza's "Applied Mechanics" the average modulus of elasticity (omitting one exceptional piece of clear pattern stock) is 1062356, and of rupture 4537. The corresponding averages of the tests 1 to 40 are 1123090 and 3906. The largest specimen in Professor Lanza's tests quoted above is 3 x 13 inches, while the largest of the West Milwaukee specimens is 10 x 16 inches, giving respective moments of resistance of $\frac{3 \times 13 \times 13}{6} = 84.5$, and $\frac{10 \times 16 \times 16}{6} = 426.67$.

Professor Lanza has also divided his tests into two classes, those which fail by rupture of extreme fibers, and those which fail by shearing in the line of the neutral axis. The writer did not make this distinction. Preferring to have one standard of strength, he adopted as the modulus of rupture the stress on extreme fibers, whether the failure was caused by breaking or shearing.

The object of the West Milwaukee experiments was to test the strength of the timber under as nearly as possible the same conditions as when in use on the bridges.

None of the specimens were selected for testing. District carpenters were ordered to send to the testing room new stringers taken without discrimination from accepted stock, and were given the numbers of trestle bridges on the road where the ages of the stringers were known, and told to take out a span from each, mark the name of the bridge on the stringers and send them to the testing room. Employees engaged in testing did not know the ages of the stringers which were taken from the bridges.

The tests lead us to the following conclusions:

First.—Green timber is not as strong as after it has seasoned.

Second.—Age and use does not weaken the timber. It preserves its strength until weakened by decay.

Third.—Knots do weaken the timber seriously, both in reducing the effective section of the beam, and in causing the fiber to be curly and cross-grained.

Fourth.—While age does not weaken the timber itself, it weakens it by season-checking.

The second and third conclusions have special bearing on pine timber from Michigan, Wisconsin and Minnesota (the writer is most familiar with timber from that section), as it is not practicable to obtain it in the quantities required for bridge construction, free of knots, and it is difficult to get it with a reasonably small number of knots. The timber in this section of country is usually small, and stringers and floor-beams are cut with the heart of the tree in the center of the stick.

In seasoning, the timber cracks from the heart in the line of least resistance, which is generally along the neutral axis of the stick. Stringers which have been long in use are, as a rule, badly season-checked along the middle of their depth.

It is also difficult to obtain timber from this section of country which is free from sap. Sap is not so much of an objection as are knots, for the former is straight-fibered and possesses strength until weakened by decay, while the latter impairs the strength of the stick from the outset.

It may be noticed in the tables that the descriptions of specimens contain the terms "butt cut," "second cut," etc. These cuts are not vouched for. Employees who were timber experts professed to be able to tell from the appearance of the sticks which cuts they were, and the entries were made accordingly. Butt cuts are usually better timber than those which come from higher up the tree, for the reason that they are less knotty. Butt cuts are also more apt to be wind shaken. The defect of wind shakes, when used as stringers, is sometimes overrated. Short grained timber is apt to break in the forest, under the effect of wind storms, while the long-fibered, straight-grained trees rock back and forth and become shaken near the butt. This class of fiber is commonly seen in the wind-shaken timber.

The number of tests made at West Milwaukee are not sufficient to determine the respective merits of white and Norway pine. The latter is more free of knots, but is more sappy than the former, and in conse-

quence decays quicker. The writer considers Norway pine, when free from sap, to be equal in quality to white pine.

Passing to tests 41 to 55, which were made at Minneapolis, these tests were made to determine the quality of a lot of new stringers, which were objected to on account of being knotty, and eight tests were made of those which were accepted and seven tests of those rejected.

These tests were made by placing a hydraulic jack on a track scale, with the specimen lying across it, and loops over the ends of specimen which were fastened to anchors in the ground each side of the scale. The scale was tested with standard test weights, and should, therefore, weigh the load accurately. The weight of the hydraulic jack and of the end loops was deducted from the scale beam reading, and the difference taken as a center load on a beam supported at the ends. It will be noticed that the average modulus of rupture of the seven accepted sticks is 6 per cent. higher than the average of the West Milwaukee tests. In these tests the difference between placing the good and bad edges in tension was noted, and the results confirmed the practice of using the best edge of a beam in tension, and placing it in the bridges with knots and other defects uppermost.

Tests 56 to 67, inclusive, are upon Douglas fir or Oregon pine, as it is differently called, and were made at the mill where the timber was sawed in the State of Washington. For this purpose a portable testing machine was necessary, and the writer, with the assistance of Mr. Hart, devised the one shown in the sketch, Plate LXXVI.

The hydraulic jack with which the stringers were broken at Minneapolis was fitted with a pressure gauge. While the Minneapolis tests were being made a large number of readings of the gauge were taken and compared with the readings of the scale beam. From these a correction was made for the friction of plunger, and the corrected gauge readings of the Washington tests may be taken as accurate, when the size of the specimens is considered. It will be noted that the fir timber is about 50 per cent. stronger than white or Norway pine. This timber is of excellent quality, free from sap, and can be obtained comparatively free from knots. Objection is made that it season-checks badly and warps in the sun. The writer has not had sufficient experience with it to know the extent of these faults, but all of the inquiries he has made have been met with replies eulogistic of the timber.

The question as to what sizes can be obtained has been answered from

several sources that it can be obtained up to 2 feet square and 100 feet long. The writer's employers have purchased a large lot of this timber for Howe truss chords and floor-beams, and the appearance of it justifies its praise.

Appended to this is the report of Mr. Hart, who represented the writer in an investigation of this timber, and which is interesting when taken in connection with tests of it.

This paper is intended as a record of the writer's notes and observations on the subject during the last two years. He has been astonished to discover what a complicated and interesting thing a plain wooden beam is, and so far from feeling that he has exhausted the subject, it is his sincere hope that other members will be sufficiently interested to pursue it.

In these days of great bridges it will not do to neglect the small ones, and the failure of a 16-foot span may wreck a train as effectually as if it were 1 600 feet.

There is nothing in this paper about the fiber stress which should be permitted. Given the loads and the strength of the material, the engineer who builds a bridge must decide the amount he will strain the latter. It may not be improper to add, as a word of warning to those who are running modern engines over their bridges, that they take note of their pine stringers and floor-beams.

It having been the writer's intention to bring to the consideration of the Society the subject of "timber trestle bridges," he ventures to add as an appendix to his paper some plans of trestle bridges and pile piers, which were prepared in his office, and upon which he invites criticism.

APPENDIX.

Report of A. J. Hart on Douglas fir.

Analysis of timber tests.

Tables of tests.

Sketch of testing machine.

Two photographs showing method of testing.

Five plans of trestle bridges and pile piers.

MINNEAPOLIS, April 5th, 1890.

Mr. ONWARD BATES, E. and S. B. and B.,

Milwaukee, Wis.:

DEAR SIR,—All of the tests I made at Tacoma, Wash., were Douglas, or Oregon fir—that is what the majority of people call it.

Unlike our white pine, it has a very coarse bark, and I should judge by the grain that it grew very rapidly. In some instances the fiber is nearly or quite ¼ inch thick and of different density. One fiber will be soft and pulpy, the other hard. This coarse-grained timber is not as strong as the finer-grained, as when it is subjected to a high strain the fiber will separate and buckle, and in some instances will pull in tension for a distance of 7 feet, as in test No. 8. It does this when green, and I do not think it would be affected in this way when dry, as it is very hard and dense when dry. I also had some trouble with the finer-grained timber, in the fiber crushing at lower edge above the jack, and shoving or doubling up. But recollect this green timber was all taken from the saw except test No. 12, which had been sawed several days and was what I called a rejected stick. A person not familiar with this fir timber would judge from the outside appearance that there were several species, as it varies in color of timber in the grain and also in the bark. I had some experience with timber in the logs at Bucoda, as they had a large quantity of logs in the river ranging in size from 2 to 7 feet in diameter and from 12 to 80 feet long. There is one feature about this timber that surprised me: that was the smooth appearance of the logs, and on opening them with a saw nine out of ten of the logs will be knotty on the inside. I picked out three logs in the river that were 3 to 4 feet in diameter and 50 to 60 feet long, and had them taken into the mill to get some sample chord sticks. We did not get one chord stick out of the three logs—all turned knotty in the heart. I was disappointed, but went back and picked out three more logs, and we got five chord sticks out of the three logs. I found by close observation that there was a perceptible swell in the log where the timber had grown over the knots. In some instances one-half of the log would be clear and the other half knotty, but on the whole I consider it very desirable timber for bridges, and I think in our climate its life will be ten to

twelve years. But this is merely my opinion based upon what they told me in Washington. They say it lasts from eight to nine years there, where the extremes between wet and dry are so great. I examined the stringers in several of their bridges, and in one instance where they said they had been in seven years they were in good condition, and I saw some stringers at Kalama that had been taken out of a bridge. The engineer said they had been in nine years. They were badly rotted at the ends where they rested on the caps.

Judging from the dry sticks I tested, this timber does not stand as great a strain when dry as when green, although the samples I had were very poor. I send you herewith a sketch of the sticks I tested, showing the condition as near as I can, as I could not take such data as I would had we been testing with our own men. We had to take men from the mill, and I rushed the tests through as fast as I could. Hoping they will be satisfactory, I remain,

Yours truly,

A. J. Hart, D. C.

Analysis of Timber Tests

Tests made at West Milwaukee April-1889- on Actual Bridge Stringers

Mark of Piece	Number of years in service	No. of Test	Kind of Timber	Nominal Section	Actual Section	Span	Breaking Load in Pounds	Modulus of Rupture in lbs pr □"	Modulus of Elasticity in lbs. per sq in		REMARKS
BRIDGE 152 LAX	4½	1	NORWAY PINE	6"×16"	5⅝"×15⅝"	14'-10½"	28,000	4880	1,567,900		Season Checks on each side about the neutral axis 1" deep on one side & 2¾" deep on the other - Failed by shearing on neutral axis.- End moved by 1 5/16" Sound throughout
"	4½	2	WHITE PINE	10"×16"	9⅞"×15¾"	14'-10½"	39,000	4080	1,120,500		Sound at breaking section, except top where it was rotted ½" under cross tie - Broke suddenly - short fracture - Coarse short grained wood - Season checked 4ft from end on one side -
BRIDGE 116 LAX	6½	3	"	7"×14"	6⅝"×13⅝"	10'-11½"	20,500	2945	1,184,000		Sound timber - season checks on both sides along the neutral axis, extending clear through for a length of 3ft on one end & about 1½" deep at other points - Failed by shearing along neutral axis ends slid by 3/16"
"	6½	4	"	7"×14"	7"×14"	10'-11½"	32,500	4670	1,076,500		Sound timber - showing no season checks - showed first sign of failing by shearing along neutral axis and finally broke at center - fibers broke in successive layers at regular intervals
"	6½	5	"	7"×14"	7"×14"	10'-11½"	18,000	2590	933,200		Sound timber, season checks at the end about 1" deep and 2ft long - same at center along neutral axis - Failed by bottom fibers breaking suddenly, and then splitting along neutral axis - end slid by ¼"
"	6½	6	"	7"×14"	7"×14"	10'-11½"	22,300	3206	915,800		Sound timber - showing small season checks - Breaking suddenly at center by bottom fibers snapping
BRIDGE 112 LAX	5½	7	"	12"×14"	12"×14"	10'-11½"	47,500	3983	1,259,000		Sound timber - season checked along neutral axis full length on one side and partly along the other about 1" deep Failed by shearing along neutral axis ends slid by 1"
—	0	8	"	10"×16"	10"×16"	14'-10½"	47,500	4966	1,308,700	line of sliding of ends	Straight, sound and free from large knots or season check Broke slowly starting along bottom fibers, and ends slid by - Butt cut
—	0	9	"	10"×16	10"×16"	14'-10½"	31,500	3294	978,500		Top cut - studded with large knots - short season checks along middle of beam - Broke suddenly at center by bottom fibers snapping - short break - coarse grained
BRIDGE 152 LAX	4½	10	"	10"×16"	10"×15⅞"	14'-10½"	32,400	3388	1,121,000		Season checks about 1" deep and 3" from bottom, extending about 4ft from one end - Started to fail by splitting along season checks, and failed by bottom fibers - breaking at center
"	4½	11	"	10"×16"	9¾"×15⅞"	14'-10½"	51,400	5376	1,430,900		Season checks on one end extending nearly across for a distance of 3ft from end. wood slightly rotten along crack. Failed by shearing along the neutral axis - end slid by ½"
BRIDGE 124 LAX SEC. 13	3½	12	"	12"×14"	12"×14"	10'-11½"	50,250	4214	1,365,000	sap 1" deep; sap	Rotted under 2 cross ties from end about ½" deep - Season checks along line of neutral axis, about 2" deep on both sides - Failed by shearing end slid by ½"
"	3½	13	"	12"×14"	12"×13⅞"	10'-11½"	49,500	4151	994,500	sap; sap	2nd cut - checks 1" deep about 3ft long at intervals along neutral axis, on both sides of beam - several large knots - Failed by breaking suddenly at center - large knot in vicinity of break

Analysis of Timber Tests - (continued)

Mark of Piece	Number of years in service	No of test	Kind of Timber	Nominal section	Actual section	span	Break'g load in Pounds	Modulus of Rupture in lbs per sq inch	Modulus of Elasticity in lbs per sq inch		REMARKS
BRIDGE 112 LAX SEC. 12	5½	14	WHITE PINE	12"×14"	12"×14"	10'-11½"	45000	3774	975,700	sap 1" deep sap	2nd Cut - Season checks at ends about ½" deep and 1ft long and one side at center 1" deep & 3' long - Broke suddenly at center by lower fibers failing - about knot
BRIDGE 152 LAX	4½	15	"	10"×16	10"×16"	14'-10½"	43500	4550	1,156,700		2nd cut - a few large knots - sound timber - short season checks along middle of beam - Broke suddenly at lower fibers on one side of center
BRIDGE 152 LAX	4½	16	NORWAY PINE	6"×16"	5⅝"×15¾"	14'-10½"	18400	3207	433,800		3rd Cut - studded with large knots, season checks 4ft long at ends and extending at short intervals along neutral axis - Broke suddenly by bottom fibers snapping
E286	4½	17	"	7"×14"	6¾"×13¾"	14'-10½"	24950	4860	1,684,100	1" sap 1" sap	Butt cut - season checks along neutral axis on both sides about ½" deep - Failed by bottom fibers snapping at center - and then shearing along neutral axis end slid by ½"
E286	4½	18	WHITE PINE	7"×14"	6¾"×14"	14'-10½"	15000	2930	822,000		2nd cut - season checks along neutral axis on both sides and at ends extending clear through - badly rotted at a point 2'-6" on one side of center at a bott hole - Broke where rotted
E166	8½	19	"	8"×15	8"×14¾"	14'-10½"	29200	4340	1,286,700		2nd cut - several knots - season checked on one side nearly through along the neutral axis - Broke at lower fibers snapping at center, and shearing along neutral axis
E166	8½	20	"	8"×15"	8"×15"	14'-10½"	24800	3570	1,281,600		Sound timber - season checked along neutral axis at intervals on both sides - Failed by shearing along neutral axis and end slid by ½"
—	0	21	NORWAY PINE	7"×15"	7⅛"×15⅛"	14'-10½"	21800	3710	1,094,500		3rd cut - studded with large knots - Broke suddenly starting at large knots - and breaking short at center
—	0	22	"	7"×15"	7⅛"×15"	14'-10½"	24400	4150	1,258,900		3rd cut - studded with large knots and broke suddenly at lower fibers at knot - broke short - slightly seasoned check
—	0	23	"	7"×15"	7"×15"	14'-10½"	24300	4130	1,188,600		2nd cut - studded with small knots - Broke slowly at center by bottom fibers snapping
—	0	24	"	7"×15"	7⅛"×15	14'-10½"	26900	4570	1,149,900		Butt cut - Clear, sound piece, free from knots or other defects - failed by bottom fibers breaking and shearing horizontally 5" below neutral axis
BRIDGE 390 C.&C.B. in ILL.	8½	25	WHITE PINE	7"×14"	7¼"×14¼"	10'-11½"	36900	5310	1,270,100		Season checked along neutral axis at intervals ½" deep both sides - Broke at bottom fibers on one side of center and sheared along neutral axis
"	8½	26	"	7"×14"	7¼"×14¼"	10'-11½"	30000	4310	1,120,500		Butt cut - season checked along neutral axis on one side about ½" deep - Failed by shearing along neutral axis
"	8½	27	"	7"×14"	7½"×14½"	10'-11½"	27000	3010	1,054,200		Butt Cut. - season checked along neutral axis at center about 1" deep on one side - broke short at center

Analysis of Timber Tests - (concluded)

Mark of Piece	Number of years in service	No. of Test	Kind of Timber	Nominal Section	Actual Section	Span	Break'g load in Pounds	Modulus of Rupture in lbs per □"	Modulus of Elasticity in lbs per sq in.		Remarks
BR. 390 C.&C. B.N. I.L.	8½	28	WHITE PINE	7"×14"	7¼"×14"	10'-11½"	35000	5030	1,277,300	line of shearing	Season checks along neutral axis ½" deep Failed by shearing End slid by + split out on annular ring growth
A314 C.&M. DIV	7½	29	"	7"×14"	7"×14"	9'-4½"	19100	2350	881,000	line of shearing	Butt cut - season checked along neutral axis at ends a distance of 3 ft. clear through in line of ring growth as shown - Failed by shearing around ring growths
"	7½	30	"	7"×14"	7"×13¾"	9'-4½"	29500	3630	712,500	1" sap	3rd cut. - sound timber - slightly season checked along neutral axis - Failed by bottom fibers breaking at knot at center.
"	7½	31	"	7"×14"	7"×13¾"	9'-4½"	31700	3900	957,000		Season checked on one end along neutral axis both sides 1" deep on one end - Failed by bottom fibers snapping on one side of center - Butt cut
"	7½	32	NORWAY PINE	7"×14"	7"×13¾"	9'-4½"	20000	2460	925,000	sap sap	Butt cut - badly season checked on one end, clear to center, and slid by on this end about ½" - Failed by shearing along neutral axis -
—	0	33	WHITE PINE	6"×16"	6"×15⅞"	14'-10½"	23600	4110	1,029,500		2nd cut - slightly season checked - Broke short by bottom fibers snapping at center - cut from large log
—	0	34	NORWAY PINE	6"×16"	6"×16½"	14'-10½"	22700	3960	1,409,900		Butt cut - season checked on half that sheared at intervals ½" deep on one side and 1" deep on other Failed by shearing along neutral axis
—	0	35	WHITE PINE	7"×14"	7"×14"	14'-10½"	12200	2380	740,000		Top cut - studded with large knots - Broke short coarse grained - Failed at lower fibers at knot on one side of center
—	0	36	"	7"×14"	7"×14¼"	14'-10½"	23000	4490	1,301,500		2nd cut - Broke short at center
—	0	37	"	7"×14"	7"×14"	14'-10½"	21200	4140	1,113,300		2nd cut - short grained Broke suddenly on one side of center
—	0	38	NORWAY PINE	7"×14"	7"×14"	14'-10½"	21350	4150	1,271,000		2nd cut - Broke very slowly, and deflected 6½" before failing by bottom fibers snapping at center
—	0	39	WHITE PINE	7"×14	6¾"×14"	14'-10½"	16200	3160	1,041,700		Top cut - studded with large knots - broke short starting at knot in bottom fibers - no horizontal shearing
—	0	40	"	7"×14"	7"×14¼"	"	22100	4310	1,231,100	sap 1" deep	2nd cut - Broke suddenly at bottom fibers at center

Moduli are figured from nominal sections

method of testing

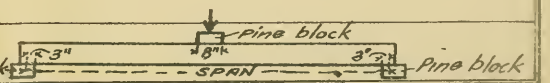


TABLE No. 1.

Moduli of Elasticity of Timber, figured from Tests made at West Milwaukee, April, 1889.

	1		2		3		4		5		6		7		8	
	6 x 16" x 14' 10½" Norway Pine.		10 x 16" x 14' 10½" White Pine.		7 x 14" x 10' 11½" White Pine.		7 x 14" x 10' 11½" White Pine.		7 x 14" x 10' 11½" White Pine.		7 x 14" x 10' 11½" White Pine.		12 x 14" x 10' 11½" White Pine.		10 x 16" x 14' 10½" White Pine.	
Load.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.
10 000	.35	1 652 900	.30	1 156 600	.25	*1 184 000	.30	987 000	.30	986 700	.30	986 700	.15	1 150 000	.25	1 388 000
12 500	.47	1 538 600	.40	1 084 400	.30	1 233 300	.35	1 055 500	.40	925 000	.40	925 000	.20	1 078 100	.30	1 445 800
15 000	.67	1 522 500	.45	*1 156 700	.45	987 400	.40	1 100 000	.50	888 000	.50	888 000	.22	1 175 500	.40	1 301 200
17 500	.65	1 557 500	.55	1 104 100	....	.....	.45	1 151 000	....	.....	.60	863 300	.30	1 006 300	.45	1 349 400
20 000	.70	*1 653 000	·65	1 067 700	....	.....	.55	1 076 300	....	.....	.60	*986 700	.35	985 700	.55	1 171 000
22 500	.80	1 627 000	.75	1 041 000	....	.....	.60	1 110 000	....	.....	....	.....	.40	970 300	.60	1 301 000
25 000	.85	1 701 000	.85	1 020 600	....	.....	.70	1 055 600	....	.....	....	.....	.40	1 078 100	.65	1 334 600
27 500	....	.....	1.00	954 300	....	.....	.75	*1 084 700	....	.....	....	.....	.45	1 054 800	.75	1 272 300
30 000	....	.....	1.1	*946 400	....	.....	.90	986 600	....	.....	....	.....	.475	1 089 500	.80	1 301 200
32 500	....	.....	1.25	902 200	....	.....	....	.....	....	.....	....	.....	.50	1 021 300	.90	1 253 000
35 000	....	.....	1.55	783 600	....	.....	....	.....	....	.....	....	.....	.55	1 097 700	.95	1 278 400
37 500	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.60	1 078 100	1.05	*1 231 000
40 000	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.65	1 061 600	1.20	1 156 700
42 500	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.70	*1 047 300	1.35	1 092 400
45 000	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	.80	970 300	1.60	976 000
47 500	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....
50 000	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....
Average	....	1 567 000	....	1 120 500	....	1 184 000	....	1 076 500	....	933 200	....	915 300	....	1 259 000	....	1 308 700

Moduli of Elasticity of Timber—(*Continued*).

Load.	9		10		11		12		13		14		15		16	
	10 x 16" x 14' 10½" White Pine.		10 x 16" x 14' 10½" White Pine.		10 x 16" x 14' 10½" White Pine.		12 x 14" x 10' 11½" White Pine.		12 x 14" x 10' 11½" White Pine.		12 x 14" x 10' 11½" White Pine.		10 x 16" x 14' 10½" White Pine.		6 x 16" x 14' 10½" Norway Pine.	
	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.
10 000	.35	991 400	.25	1 388 000	.20	1 735 000	.10	1 725 000	.15	1 150 000	.15	1 150 000	.30	1 156 700	.80	433 800
12 500	.45	964 000	.40	1 084 400	.30	1 445 800	.15	1 437 500	.20	1 078 100	.25	862 500	.35	*1 239 300	1.00	*433 800
15 000	.55	946 400	.50	1 031 000	.35	1 487 100	.20	1 283 700	.25	1 035 000	.25	1 035 000	.45	1 156 700	1.25	416 400
17 500	.60	1 012 100	.55	1 214 500	.40	1 518 100	.20	1 509 400	.30	1 006 200	.30	1 006 200	.50	1 214 500	....	.....
20 000	.70	*991 400	.65	1 067 700	.55	1 261 800	.25	1 380 000	.35	985 600	.35	985 600	.55	1 261 800	....	.....
22 500	.85	918 500	.75	1 041 000	.60	1 301 200	.25	1 552 500	.40	970 300	.40	970 300	.60	1 301 300	....	.....
25 000	.90	*963 900	.85	1 020 600	.65	1 334 600	.30	1 437 500	.45	958 300	.45	958 300	.70	1 239 300	....	.....
27 500	1.00	954 300	.925	*1 031 600	.70	1 363 200	.35	1 357 300	.50	948 700	.50	950 100	.80	1 192 800	....	.....
30 000	1.30	800 700	1.05	991 400	.75	*1 388 000	.40	1 293 700	.55	941 000	.55	941 000	.90	1 156 700	....	.....
32 500	....	.....	....	.....	.85	1 326 800	.45	1 245 800	.60	934 400	.60	934 400	1.00	*1 127 700	....	.....
35 000	....	.....	...	.....	.90	1 349 400	.50	1 207 500	.65	929 000	.65	939 400	1.125	1 079 500	....	.....
37 500	....	.....	....	.....	1.05	1 239 300	.55	1 176 500	.675	*958 300	.675	*958 300	1.20	1 084 400	....	.....
40 000	....	.....	....	.....	1.15	1 207 000	.60	1 150 000	.75	920 000	.75	920 000	1.475	941 000	....	.....
42 500	....	.....	...	.....	1.225	1 204 000	.625	*1 173 000	.80	916 400	.80	916 400	....	.....	....	.....
47 500	....	.....	....	.....	1.326	1 178 500	.70	1 108 900	.85	913 200	....	.....	....	.....	....	.....
50 000	....	.....	....	.....	1.45	1 136 700	.80	1 092 500	1.10	840 000	....	.....	....	.....	....	.....
Average	....	978 500	....	1 121 000	....	1 430 900	....	1 365 000	....	994 500	....	975 700	....	1 156 700	....	433 800

Moduli of Elasticity of Timber—(*Continued*).

Load.	17 7 x 14" x 14' 10½" Norway Pine.		18 7 x 14" x 14' 10½" White Pine.		19 8 x 15" x 14' 10½" White Pine.		20 8 x 15" x 14' 10½" White Pine.		21 7 x 15" x 14' 10½" Norway Pine.		22 7 x 15" x 14' 10½" Norway Pine.		23 7 x 15" x 14' 10½" Norway Pine.		24 7 x 15" x 14' 10½" Norway Pine.	
	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity	Deflection in inches.	Modulus of Elasticity.
10 000	.40	1 850 000	.90	*822 000	.35	1 506 000	.40	1 316 500	.55	1 094 500	.45	1 338 000	.5	1 204 000	.5	1 204 000
12 500	.55	1 681 800	1.2	771 000	.55	1 197 000	.50	1 316 500	.7	*1 075 000	.60	1 254 200	.65	1 157 700	.65	1 157 700
15 000	.70	1 585 600	....	.....	.65	1 215 000	.60	1 316 500	.9	1 003 200	.75	1 204 000	.75	1 204 000	.8	1 128 800
17 500	.80	1 619 000	....	.....	.75	1 228 800	.70	1 316 500	1.15	916 100	.85	1 239 400	.90	1 170 600	.95	1 109 000
20 000	.90	1 644 500	....	.....	.80	*1 316 500	.85	1,239 000	....	.....	1.00	*1 204 000	1.05	1 146 700	1.15	*1 047 000
22 500	1.2	1 387 500	....	.....	.90	1 316 500	1.00	1 184 800	....	.....	1.15	*1 178 000	1.35	1 200 300	1.35	1 003 000
25 000	....	.....	....	.....	1.05	1 253 800	....	.....	....	.....	....	.....	....	.....	1.35	1 114 000
Average	....	1 684 100	....	822 000	....	1 286 700	....	1 281 600	....	1 094 500	....	1 258 900	....	1 188 600	....	1 149 900

Load.	25 7 x 14" x 10' 11½" White Pine.		26 7 x 14" x 10' 11½"		27 7 x 14" x 10' 11½" White Pine.		28 7 x 14" x 10' 11½" White Pine.		29 7 x 14" x 9' 4½" White Pine.		30 7 x 14" x 9' 4½" White Pine.		31 7 x 14" x 9' 4½" White Pine.		32 7 x 14" x 9' 4½" White Pine.	
10 000	.2	1 480 000	.25	1 184 000	.25	1 184 000	.20	1 480 000	.20	925 000	.25	740 000	.15	1 233 300	.20	925 000
12 500	.3	1 233 000	.35	1 057 000	.35	1 057 000	.30	1 233 000	.25	925 000	.30	770 000	.25	925 000	.25	925 000
15 000	.35	1 268 600	.35	*1 268 600	.40	1 110 000	.35	1 268 600	.35	793 000	.40	693 000	.30	925 000	....	.....
17 500	.4	1 295 000	.45	1 151 000	.50	1 036 000	.40	1 295 000	.85	*381 000	.50	647 000	.35	925 000	....	.....
20 000	.45	1 315 500	.5	*1 184 000	.60	986 700	.45	1 315 500	....	.....	.50	*740 000	.40	925 000	....	.....
22 500	.5	1 337 000	.60	1 110 000	.70	951 400	.55	1 211 000	....	.....	.60	693 000	.45	.925 000	....	.....
25 000	.6	1 233 000	.70	1 055 600	.75	*1 986 600	.65	1 138 300	....	.....	.70	660 700	.55	.841 000	....	.....
27 500	.7	1 163 000	.80	1 017 500	....	.....	.65	*1 229 500	....	.....	....	.....	.60	*.847 900	....	.....
30 000	.8	1 110 000	....	.....	....	.....	.80	1 110 000	....	.....	....	.....	.80	.693 700	....	.....
32 500	.9	*1 069 000	....	.....	....	.....	.90	1 057 800	....	.....	....	.....	....	.....	....	.....
35 000	.10	1 016 000	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....	....	.....
Average	....	1 270 100	....	1 120 500	....	1 054 200	....	1 277 300	....	881 000	....	712 500	....	957 000	....	925 000

Moduli of Elasticity of Timber—(*Concluded*).

Load.	33		34		35		36		37		38		39		40	
	6 x 16" x 14' 10½" White Pine.		6 x 16" x 14' x 10½" Norway Pine.		7 x 14" x 14' x 10½" White Pine.		7 x 14" x 14' x 10½" White Pine.		7 x 14" x 14' x 10½" White Pine.		7 x 14" x 14' x 10½" Norway Pine.		7 x 14" x 14' x 10½" White Pine.		7 x 14" x 14' x 10½" White Pine.	
	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.	Deflection in inches.	Modulus of Elasticity.
10 000	.55	1 051 800	.40	1 446 200	1.0	*740 000	.55	1 345 400	.65	1 138 400	.55	1 345 400	.70	1 055 600	.60	1 233 300
12 500	.70	1 033 000	.50	1 446 200	...	.....	.70	1 321 400	.85	1 088 200	.75	1 233 300	.90	1 027 800	.75	1 226 600
15 000	.85	1 020 900	.60	1 446 200	...	.....	.85	1 306 000	.95	*1 168 400	.90	1 233 300	1.15	*965 200	.90	1 233 300
17 500	1.00	1 012 400	.75	1 349 800	...	.....	1.05	1.233 300	1.15	1 126 100	1.125	*1 151 100	....	.....	1.05	*1 233 300
20 000	1.15	*1 006 100	.85	1 361 200	...	.....	1.2	*1 233 300	1.5	986 700	1.45	1 020 700	....	.....	1.30	1 138 500
22 500	1.45	900 000	...	.....	...	.....	....	.....	....	.....	....	.....	....	.....	....	.....
Average	.....	1 029 500	...	1 409 900	...	740 000	....	1 301 500	....	1 113 300	....	1 271 000	....	1 041 700	....	1 231 100

Note.—The average modulus given is the average of the moduli below the first permanent set, and where such set occurred at the first load, the modulus figured from such deflection, after deducting the permanent set, was used.

* Indicates the load under which the first permanent set occurred.
In columns where two asterisks are shown, the permanent set did not increase between those two points.

Modulus of elasticity was figured by the following formula:

$$E = \frac{W \times l^3}{d \times 48 \times I} \text{ where } \begin{cases} W = \text{load applied at center.} \\ l = \text{length of span.} \\ d = \text{deflection.} \\ I = \text{moment of inertia.} \end{cases}$$

General average of forty tests, computed from the above averages, = 1 123 090.

TABLE No. 2.

Comparison of Results of Timber Tests made at West Milwaukee, April, 1889.

	Number of Tests.	Modulus of Rupture.			Modulus of Elasticity.		
		Highest.	Lowest.	Average.	Highest.	Lowest.	Average.
Years in service 0	14	4 966	2 380	3 966	1 409 900	740 000	1 151 220
" " 3½	2	4 214	4 151	4 182	1 365 000	994 500	1 179 750
" " 4½	8	5 376	2 930	4 159	1 684 100	433 800	1 167 110
" " 5½	2	3 983	3 774	3 878	1 259 000	975 700	1 117 350
" " 6½	4	4 670	2 590	3 353	1 184 000	915 800	1 027 380
" " 7½	4	3 900	2 350	3 085	957 000	712 590	868 880
" " 8½	6	5 310	3 010	4 262	1 286 700	1 054 200	1 215 070
	40	5 376	2 350	3 906	1 684 100	712 500	1 123 090
White pine	30	5 376	2 350	3 872	1 430 900	712 500	1 098 000
Norway pine	10	4 880	2 460	4 008	1 684 100	433 800	1 198 360
Cross-section 6 x 16"	4	4 880	3 207	4 039			
" 10 x 16"	6	5 376	3 294	4 276			
" 7 x 14"	20	5 310	2 350	3 692			
" 12 x 14"	4	4 214	3 774	4 030			
" 7 x 15"	4	4 570	3 710	4 140			
" 8 x 15"	2	4 340	3 570	3 955			

TABLE No. 3.

ANALYSIS OF TESTS OF WHITE PINE TIMBER.—TESTS MADE AT MINNEAPOLIS, DECEMBER, 1889, ON ACTUAL BRIDGE STRINGERS.—ALL NEW TIMBER.—LENGTH OF SPAN FOR TESTING, 15′ 10″, CENTER TO CENTER SUPPORTS.

No. of Test.	Nominal Section.	Actual Section.	Last Deflection before Failure.	Breaking Load in Pounds.	Modulus of Rupture, Pounds per Square Inch.	Remarks
41	8 x 16″	Not taken.	1″	19 570	2 723	This was a very poor and knotty stick. Worst edge in tension. Broke 2 feet to one side of center bearing.
42	"	"	$1\frac{7}{16}$″	27 170	3 781	About same quality of stick as No. 41. Best edge in tension. Broke in center.
43	"	"	$2\frac{11}{16}$″	36 870	5 131	Very knotty stick, but knots were sound, and good fiber. Best edge in tension. Broke one foot to one side of center bearing.
44	"	"	$1\frac{1}{2}$″	22 720	3 162	Knots about same as No. 43, but fiber was short grained. Worst edge in tension. Broke in center.
45	"	"	$1\frac{15}{16}$″	27 520	3 830	Good fiber with some knots. Best edge in tension. Broke near center.
46	"	"	$1\frac{5}{8}$″	25 270	3 517	Short fiber with some knots. Worst edge in ten ion. Broke short in center.
47	"	"	$2\frac{3}{8}$″	32 920	4 581	Good quality with straight fiber.
48	"	"	$2\frac{1}{4}$″	35 770	4 978	Good quality with straight fiber. Sap on one edge. Sap edge in tension.
49	"	$7\frac{3}{4}$ x $16\frac{1}{8}$″	$1\frac{3}{4}$″	26 520	3 690	Poor stick. Rejected for knots. Best edge in tension.
50	"	8 x $16\frac{1}{4}$″	$\frac{7}{8}$″	15 520	2 160	Very knotty. Large knots run from center of side to corner of stringer. Rejected for knots. Worst edge in tension. Broke at knot, 16 inches from center bearing.
51	"	$8\frac{1}{8}$ x $16\frac{1}{8}$″	$1\frac{1}{4}$″	21 920	3 050	Rejected for knots. Best edge in tension. Broke at knot, 15 inches from center bearing.
52	"	8 x $16\frac{3}{8}$″	$1\frac{13}{16}$″	30 020	4 178	Rejected. Small knots but no bad ones. Best edge in tension. Break commenced at knot near center bearing.
53	"	$8\frac{1}{4}$ x $15\frac{3}{4}$″	$1\frac{1}{2}$″	20 020	2 786	Rejected. Best edge in tension. Broke at a bad knot near middle of stick.
54	"	$8\frac{1}{4}$ x $16\frac{1}{8}$″	$2\frac{1}{4}$″	29 820	4 150	Rejected. Best edge in tension. Broke short at center.
55	"	8 x 16″	$1\frac{9}{16}$″	Not broken at 34 350	f. for 34 350 =4 780	Good quality. Best edge in tension. Anchorage at end of stick failed, and it was not tested to destruction.

MODULI OF RUPTURE.

Remarks		Highest	Lowest	Average
	7 Accepted.	5 131.	3 162.	4 140.
	7 Rejected.	" 4 178.	" 2 160.	" 3 248.
	14 Exclusive of No. 55.	" 5 138.	" 2 160.	" 3 694.

TABLE No. 4.

Analysis of Tests of Douglas Fir Timber.—Tests Made in Washington Territory, March, 1890, on Actual Bridge Stringers.

Number of Years in Service.	No. of Test.	Kind of Timber.	Nominal Section.	Actual Section.	Span.	Breaking Load in Pounds.	Modulus of Rupture in Pounds per Square Inch.	Last Deflection before Failure.	Remarks.
0	56	Douglas Fir.	6 x 16″	6⅛ x 15¾″	18′ 0″	25 297	5 336	2⅞″	Green stick, quite knotty. Fiber compressed at center over jack and lost all resistance at 25 297 lbs. Did not fracture at tension edge.
0	57	"	6 x 14″	6 x 14″	15′ 9″	29 468	7 104	2⅛″	Green timber, with heart of stick on one edge which was quite knotty, heart edge in tension. Broke at center fracture—commenced at knot in tension edge and split both ways.
0	58	"	6 x 14″	6 x 14″	11′ 0″	38 733	6 522	1½″	Green, soft stick, clear of knots. Fibers commenced to compress or buckle at 29 000 lbs., and lost all resistance at 39 111 lbs.
0	59	"	6 x 14″	6 x 14″	15′ 0″	26 689	6 127	1½″	Old dry stick, picked up in yard, had been rejected on account of knots. Broke suddenly at the center.
0	60	"	8 x 16″	8 x 16″	19′ 0″	45 056	7 524	2½″	Green, fine-grained, good average stick. Broke at the center and split each way about 6 inches from the top. Top fibers compressed over jack to some extent.
6	61	"	9 x 16″	9 x 16″	19′ 0″	25 297	3 755	1½″	Old and dry stick, had been in bridge six years. Was furnished by Engineer of N. P. Ry. Very poor sample. Broke at center and split each way.
3	62	"	8 x 16″	8 x 16″	16′ 0″	39 660	5 578	1¾″	Dry stringer, was taken from N. P. Ry. bridge, badly cut with spikes, otherwise fair stick. Fractured at center and split nearly full length—best edge in tension.
0	63	"	8 x 16″	8⅛ x 16″	15′ 9″	54 501	7 544	2 1/16″	Average quality, fine-grained, small knots, green timber, sawed for testing. Fractured at center and split.
0	64	"	8 x 16″	8 x 16″	19′ 0″	32 248	5 385	2⅛″	Old dry stick and somewhat decayed. Fractured at center and split in all directions—knotty.
0	65	"	9 x 16″	9¼ x 16″	19′ 0″	38 733	5 750	2″	Green stick, from the saw, good, fair sample, free from knots. Fractured at center and split both ways.
3	66	"	9 x 16″	8¾ x 16″	19′ 0″	35 490	5 268	2½″	Old, dry and knotty stick, had been sawed about three years. Fractured at center and split each way.
0	67	"	8 x 14″	8 x 14″	19′ 0″	16 497	3 597	1⅞″	Green stick, poor quality, had been sawed several days, was knotty and cross-grained. Broke suddenly 2½ feet to one side of center.

Moduli of Rupture.

12 tests (No. 56 to No. 67 inclusive). Highest, 7 544 lbs. Lowest, 3 597 lbs. Average, 5 791 lbs.
10 " (omitting No. 61 and No. 67) " 7 544 lbs. " 5 268 lbs. " 6 214 lbs.

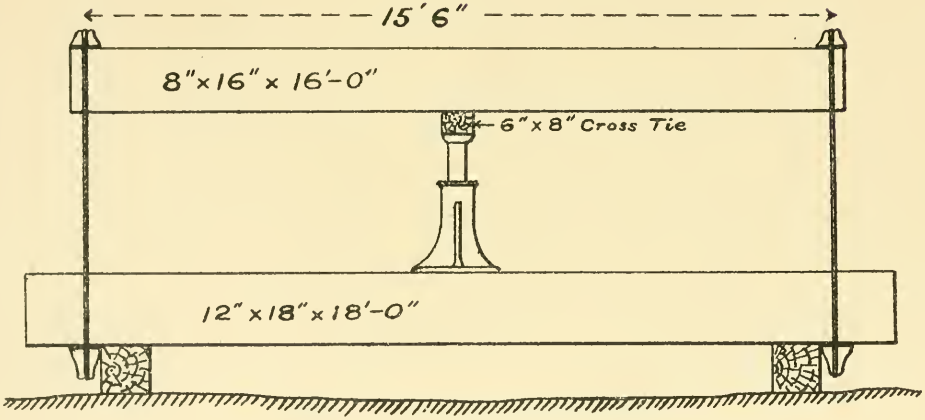
DISCUSSION.

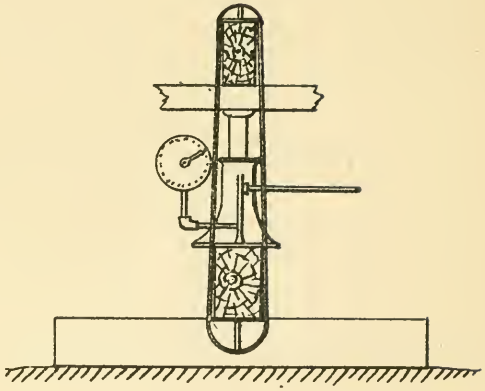
C. Palmer, M. Am. Soc. C. E.—These tables are extremely valuable. One could wish that they embraced a greater range of sizes and lengths, but the labor of preparing them must have been great, and they cover a reasonable amount of ground. With a little patience and judgment other sizes can be interpolated. I find a substantial agreement between the results and the formulas Boller gives, on pages 105, 106 and 107 of his "Highway Bridges," turning center into distributed loads, and using a factor of 4 to 6. But while all this assists our judgment, very much remains to be considered in the preparation of efficient plans. The deflections for loads corresponding to that of an engine with 60 000 pounds on drivers are from one-quarter to one-half inch. This imparts an undulatory motion to the load, resulting in an impact which is practically impossible to compute. It often happens too that the track on the embankment approaching a bridge is so out of surface that the engine literally bounds on to it. These things suggest a wide margin for safety.

There is but little criticism to make on the plans. Practice varies, and there are a good many good plans. These are of ample strength and are well sway-braced, but I prefer 2½ to 3 inch batter for the outside posts. I like short bolsters under ends of stringers. They practically shorten the span at each end to the amount of their depth, and increase the stringer bearings. Mr. Bates must have observed instances where stringers bearing but 6 inches on caps have lost a good portion of that by the decay of a sappy corner on the cap.

Referring to the joint, *A*, in the two-story bent, I would either raise the longitudinal tie one foot above the timber girt, or place it on the inside of the post, to get rid of the receptacle for moisture on the outside of the post at its foot.

I notice the outside track stringers are fastened to the caps with bridge bolts. That is well enough if the piles are so well driven that the bridge will never need surfacing. Railroad companies can get such work done by their regular bridge force, but it takes more efficient watching than I could ever secure to get it done by contractors.





TIMBER TESTING MACHINE.

BILL OF MATERIAL—

1 30 Ton Hydraulic Jack (Non-Freezing) with Pressure Gauge attached, registering loads up to 50,000 pounds.

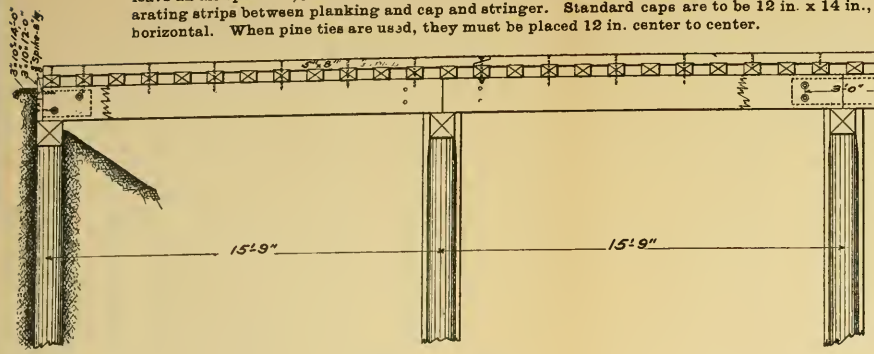
2 Stirrups of 1 inch diameter iron.

2 Cast Washers for Stirrups with bearing 6" x 8" on Stringer.

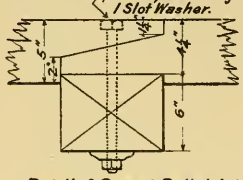
2 " " " " " " 6" x 12" " Sill.

Track Scales to be tested by weight; then Gauge to be corrected by Track Scales.

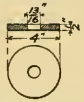
NOTE.—Ends of Stringers must be so cut that they will abut against each other for 2 in. down from the top and leave an air space of ⅜ in. between their ends at bottom. At bank bents a 2 in. air space must be provided by separating strips between planking and cap and stringer. Standard caps are to be 12 in. x 14 in., with the 14 in. side horizontal. When pine ties are used, they must be placed 12 in. center to center.



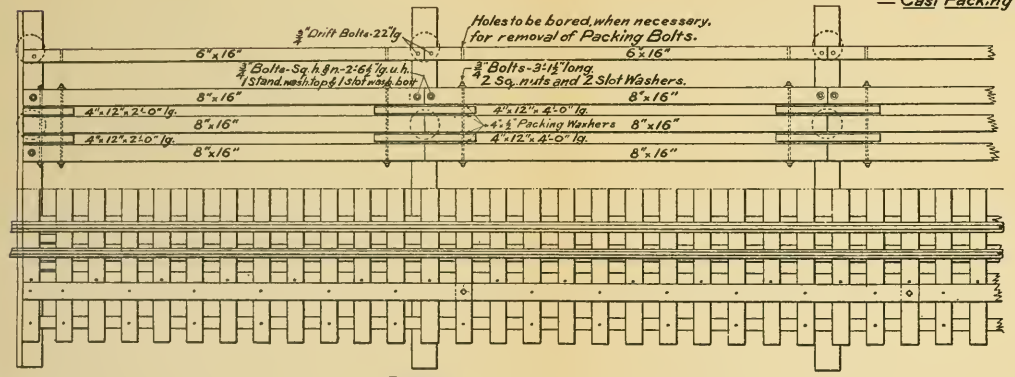
—ELEVATION.—



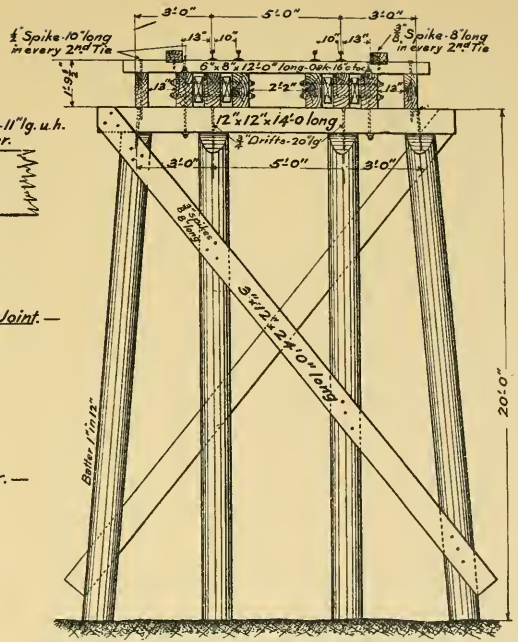
—Detail of Guard Rail Joint.—



—Cast Packing Washer.—



—PLAN.—



—NOTE— No Sway Bracing on Bents less than 6 feet high.—

—SECTION.—

—Pile Bents up to 20 ft. High.—

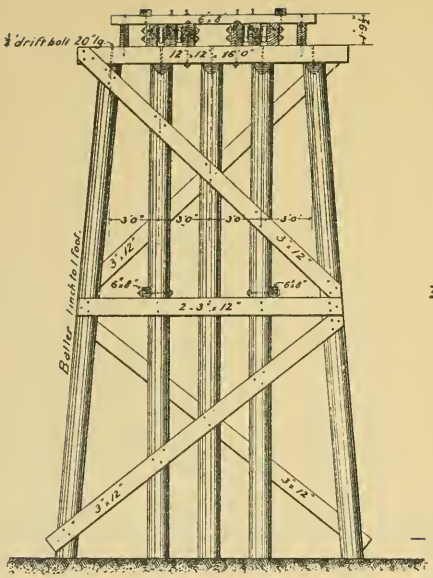
—C., M. & ST. P. RY.—

—BRIDGE & BUILDING DEP'T.—

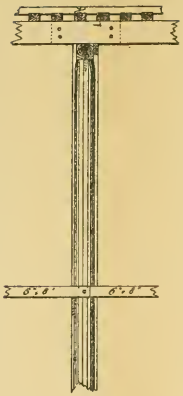
—Standard Pile Bent Trestle.—

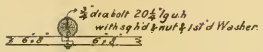
—Milwaukee, Feb., 1889—

SCALE

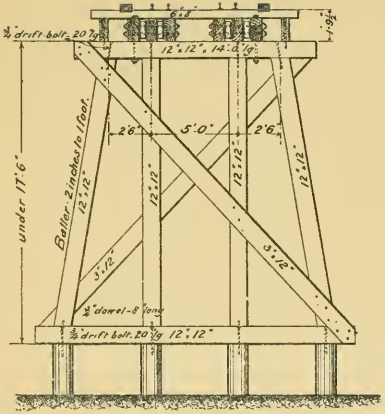


— Pile Bent over 20 ft high —

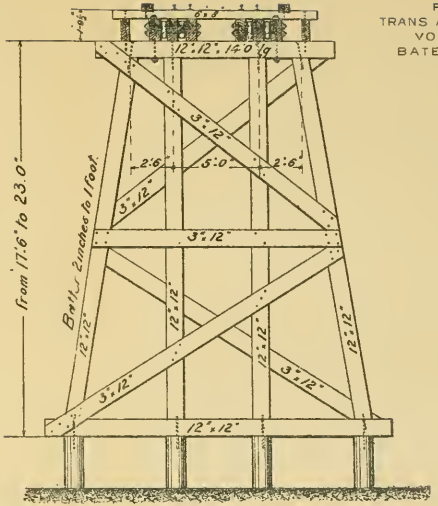




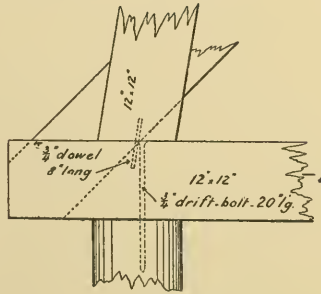
— Detail for fastening Girts —



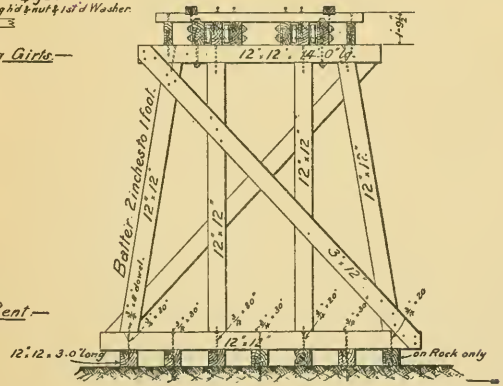
— Framed Bent under 17'-6" —



— Framed Bent from 17'-6" to 23'-0" —



— Detail for Framed Bent —



— Framed Bent resting on Sills —

— C. M. & St. P. Ry. —

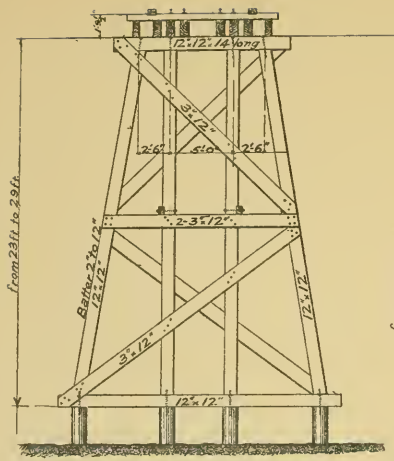
— Bridge & Building Dep't. —

Standard Pile & Frame Bents.

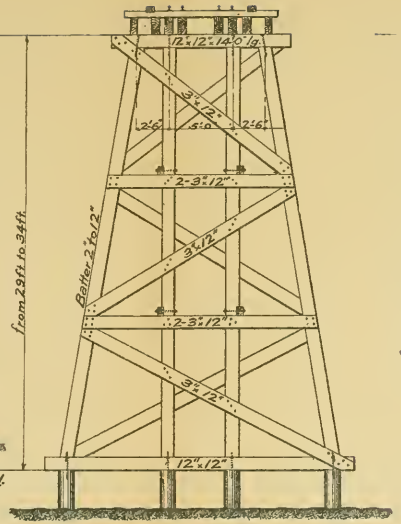
— Milwaukee, Feb 1889 —

Scale

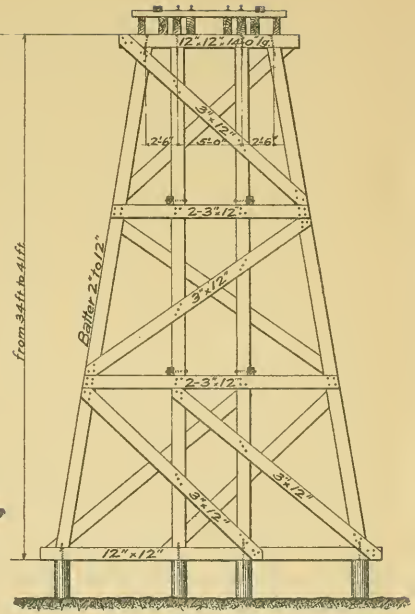
PLATE LXXIX
TRANS. AM SOC CIV ENGRS
VOL XXIII No 455
BATES ON BRIDGES.



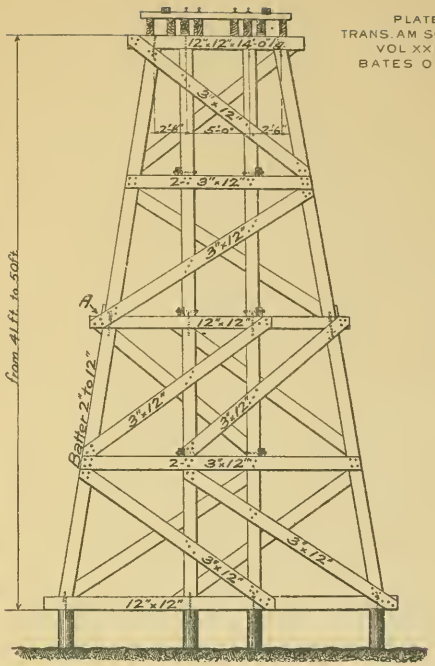
FRAMED BENTS FROM 23FT TO 29FT HIGH.



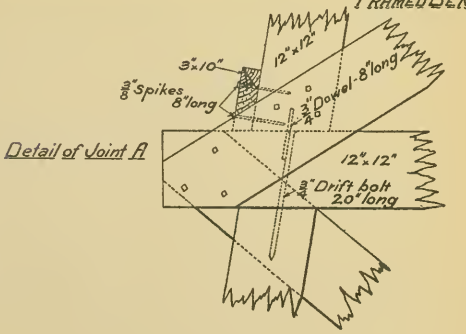
FRAMED BENTS FROM 29FT TO 34FT HIGH.



FRAMED BENTS FROM 34FT TO 41FT HIGH.



FRAMED BENTS FROM 41FT TO 50FT HIGH.



Detail of Joint A

—C., M. & ST. P. RY.—

—BRIDGE & BUILDING DEPT.—

—Standard Framed Bent Trestle—

—Milwaukee, March, 1889.—

SCALE

PLATE LXXX
TRANS. AM. SOC. CIV. ENGRS
VOL. XXIII. No 455.
BATES ON BRIDGES.

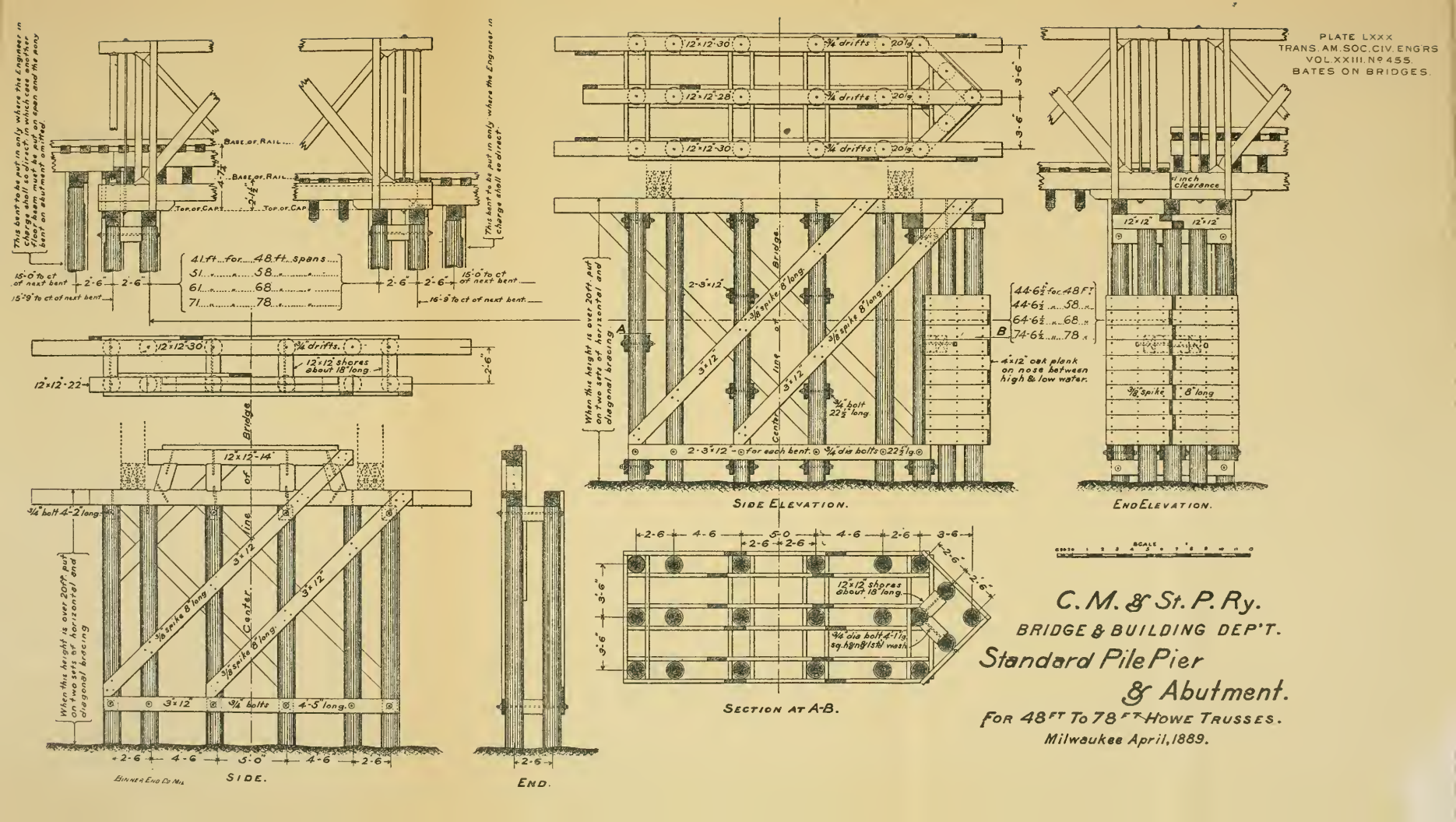
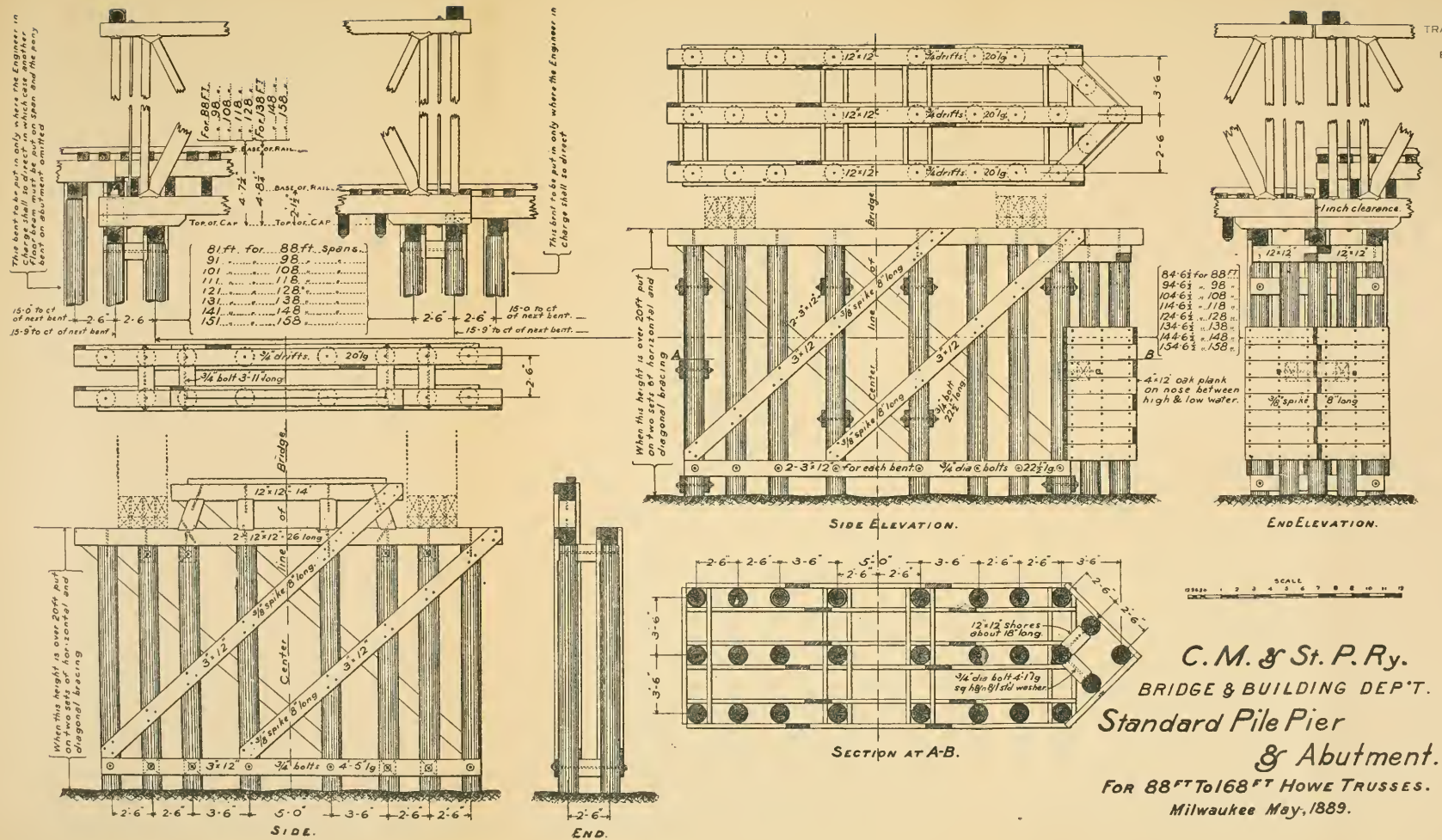


PLATE LXXXI
TRANS. AM. SOC. CIV. ENGRS.
VOL XXIII No 455.
BATES ON BRIDGES.



C. M. & St. P. Ry.

BRIDGE & BUILDING DEP'T.

Standard Pile Pier & Abutment.

FOR 88 FT TO 168 FT HOWE TRUSSES.

Milwaukee May, 1889.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

456.

(Vol. XXIII.—December, 1890.)

COMPONENTIAL TRUSSES FOR TRAVELING CRANE.

By HENRY B. SEAMAN, M. Am. Soc. C. E.

There was recently constructed at the marble yard of Messrs. John Baird & Sons, Philadelphia, an iron trestle, complying with unusual conditions. Since its design possesses some peculiar features of engineering interest, the following description is submitted to the members of the Society.

The marble yard mentioned is located on the Schuylkill River, just south of Walnut Street, and extends from 24th Street to the wharf line, a distance of about 320 feet. It has on 24th Street an extensive saw mill, and the yard space between the mill and the river is used for the storage of blocks of marble. These blocks are conveyed from the river, and distributed over the yard, or carried into the mill as the case may require, by a traveling crane which moves on two elevated rails, one at each side of the yard.

When the Baltimore and Ohio Railroad extended its lines into the City of Philadelphia, it crossed Messrs. Baird & Sons' property, and threatened to seriously injure their business by separating the mill from the yard. The elevation of the grade of the railroad was such as to permit of only 2 feet 5½ inches for the depth of any overhead structure, while the width of the right of way to be spanned was over 60 feet. At that time, shallow box girders were constructed, carrying the stringer and the elevated rail over the right of way, and by this means the traveler was permitted to pass from the yard into the mill.

After the railroad had been in active operation for about two years, however, its yard room adjacent to Chestnut Street Station proved insufficient for its increasing business, and, in order to reach additional tracks south of Walnut Street, it was proposed to increase the width of the right of way across the yard to 100 feet, and to support the elevated tracks over the railroad, as before, on iron girders. The maximum allowable depth of 2 feet 5½ inches, with a clear span of 100 feet, made the problem a serious one. The shape of the traveler, extending 12 inches beyond the rail, rendered the use of a deep girder, outside of the stringer, objectionable on account of the lateral moment produced. The construction of the mill was such as to prevent the further elevation of the track; and the required clearance beneath, with a proper limit of deflection, made the use of a deck girder, as before, impracticable.

The work was placed in the hands of Messrs. Wilson Bros. & Co., of Philadelphia, for whom the writer was Principal Assistant Engineer, and the plan adopted, as promising the most satisfactory results with the greatest economy, was the one herein described.

The general design, as may be seen by the accompanying photograph, Plate LXXXII, was such as to place the supporting girders entirely outside of the stringers, and at the same time to provide for all lateral moments produced in the structure. In detail, the vertical load was resolved into two components—horizontal and oblique—and these components were each carried by a separate web system. The oblique system was of sufficient depth to give vertical stiffness to the truss, and was placed at such an inclination as to avoid the end of the traveler—the horizontal system was so constructed that its outer chord would come directly under that of the oblique system. These componential web systems were united at their apex by a common chord; and their remaining chords, on the outside of the truss, were, in turn, united by a vertical

system. Thus was constructed a truss with three chords and three web systems.

The componential systems carry the live and dead loads applied at their apex, while the vertical system carries the dead load applied at its chords. The top and bottom chords of the vertical system act as the chords proper of the truss, while the chord at the apex under the applied loads serves to transfer the strains from one componential system to the other, and at the same time acts as a stringer to carry the live load between panel points.

The action of the forces in the truss is an interesting feature. Owing to the peculiar form of construction, there are at each end of the truss two distinct vertical reactions—one at each supporting column. The reaction of the outside column, under the vertical system, is due entirely to the strains in that system; and since its purpose is merely to retain the two chords in position, the reaction is equal to one-half of the weight of these chords, and is, therefore, a constant quantity. Having obtained the reaction of the outside column, that of the inside column, under the stringer, is found by subtraction—or, since the former is due to the dead load of the two outside chords, the latter is due to the dead load of the stringer chord, and to the live load. In each case the weight of the web systems is assumed as being carried equally by the two adjacent chords.

In tracing the effects of these reactions through the truss, reference may be made to the outline diagram shown on Plate LXXXIV.

The reaction of column "*a*," under the vertical system, is produced directly by that system, and may be readily traced through it, in the usual manner. The reaction of column "*b*" is equally definite in its action, though not quite so quickly traced—the force is first resolved into the two components of the adjacent web systems, and these components are then treated as original reactions. The oblique component is resisted directly by the end post of the oblique system "c_1 b_1," while the horizontal component is again resolved, and resisted by the end diagonal of the horizontal system "b_1 $a_{1\frac{1}{2}}$" and by the end panel of the stringer chord "b_1 $b_{1\frac{1}{2}}$."

Thus the strains are traced through the entire system, with the following results:

WEB STRAINS.

Vertical System.

	Dead Load. Pounds.		Dead Load. Pounds.
$c_1\ a_1$ =	+ 8 400	$c_1\ a_2$ =	— 11 700
$c_2\ a_2$ =	+ 7 200	$c_2\ a_3$ =	— 9 100
$c_3\ a_3$ =	+ 5 400	$c_3\ a_4$ =	— 6 500
$c_4\ a_4$ =	+ 3 700	$c_4\ a_5$ =	— 3 900
$c_5\ a_5$ =	+ 1 900	$c_5\ a_6$ =	— 1 300
$c_6\ a_6$ =	+ 1 050		

Oblique System.

	Dead Load. Pounds.		Live Load. Pounds.		Total. Pounds.
$c_1\ b_1$ =	+ 8 800	+	29 100	=	+ 37 900
$c_2\ b_2$ =	+ 6 800	+	25 700	=	+ 32 500
$c_3\ b_3$ =	+ 4 900	+	22 400	=	+ 27 300
$c_4\ b_4$ =	+ 2 900	+	19 000	=	+ 21 900
$c_5\ b_5$ =	+ 1 000	+	15 700	=	+ 16 700
$c_6\ b_6$ =	+	+	12 300	=	+ 12 300

	Dead Load. Pounds.		Live Load. Pounds.		Total Pounds.
$c_1\ b_2$ =	— 12 300	—	40 700	=	— 53 000
$c_2\ b_3$ =	— 9 600	—	36 000	=	— 45 600
$c_3\ b_4$ =	— 6 800	—	31 300	=	— 38 100
$c_4\ b_5$ =	— 4 100	—	26 600	=	— 30 700
$c_5\ b_6$ =	— 1 400	—	21 900	=	— 23 300
$c_6\ b_7$ =	.....	—	15 800	=	— 15 800
$c_7\ b_8$ =	.....	—	8 400	=	— 8 400
$c_8\ b_9$ =	.....	—	1 000	=	— 1 000

Horizontal System.

		Dead Load. Pounds.		Live Load. Pounds.		Total. Pounds.
$b_{1\frac{1}{2}}\ a_{1\frac{1}{2}}$ =	$b_2\ a_2$ =	+ 3 900	+	13 000	=	+ 16 900
$b_{2\frac{1}{2}}\ a_{2\frac{1}{2}}$ =	$b_3\ a_3$ =	+ 3 100	+	11 500	=	+ 14 600
$b_{3\frac{1}{2}}\ a_{3\frac{1}{2}}$ =	$b_4\ a_4$ =	+ 2 200	+	10 000	=	+ 12 200
$b_{4\frac{1}{2}}\ a_{4\frac{1}{2}}$ =	$b_5\ a_5$ =	+ 1 300	+	8 500	=	+ 9 800
$b_{5\frac{1}{2}}\ a_{5\frac{1}{2}}$ =	$b_6\ a_6$ =	+ 400	+	7 000	=	+ 7 400

		Dead Load. Pounds.		Live Load. Pounds.		Total. Pounds.
$b_1\ a_{1\frac{1}{2}}$ =	$b_{1\frac{1}{2}}\ a_2$ =	— 5 900	—	19 400	=	— 25 300
$b_2\ a_{2\frac{1}{2}}$ =	$b_{2\frac{1}{2}}\ a_3$ =	— 4 600	—	17 100	=	— 21 700
$b_3\ a_{3\frac{1}{2}}$ =	$b_{3\frac{1}{2}}\ a_4$ =	— 3 300	—	14 900	=	— 18 200
$b_4\ a_{4\frac{1}{2}}$ =	$b_{4\frac{1}{2}}\ a_5$ =	— 2 000	—	12 700	=	— 14 700
$b_5\ a_{5\frac{1}{2}}$ =	$b_{5\frac{1}{2}}\ a_6$ =	— 700	—	10 400	=	— 11 100
$b_6\ a_{6\frac{1}{2}}$ =	$b_{6\frac{1}{2}}\ a_7$ =	...	—	7 500	=	— 7 500
$b_7\ a_{7\frac{1}{2}}$ =	$b^{7\frac{1}{2}}\ a_8$ =	...	—	4 000	=	— 4 000
$b_8\ a_{8\frac{1}{2}}$ =	$b_{8\frac{1}{2}}\ a_9$ =	...	—	500	=	— 500

Stringer Chord.

		Dead Load. Pounds.		Live Load. Pounds.		Total. Pounds.
$b_1\ b_{1\frac{1}{2}}$	=	+ 4 330	+	14 300	=	+ 18 630
$b_{1\frac{1}{2}}\ b_2$	=	+ 8 660	+	28 600	=	+ 37 260
$b_2\ b_{2\frac{1}{2}}$	=	+ 3 370	+	12 650	=	+ 16 020
$b_{2\frac{1}{2}}\ b_3$	=	+ 6 740	+	25 300	=	+ 32 040
$b_3\ b_{3\frac{1}{2}}$	=	+ 2 400	+	11 000	=	+ 13 400
$b_{3\frac{1}{2}}\ b_4$	=	+ 4 800	+	22 000	=	+ 26 800
$b_4\ b_{4\frac{1}{2}}$	=	+ 1 450	+	9 350	=	+ 10 800
$b_{4\frac{1}{2}}\ b_5$	=	+ 2 900	+	18 700	=	+ 21 600
$b_5\ b_{5\frac{1}{2}}$	=	+ 480	+	7 700	=	+ 8 180
$b_{5\frac{1}{2}}\ b_6$	=	+ 960	+	15 400	=	+ 16 360

Chord Strains.

		Dead Load. (Vert. System.) Pounds.		Dead Load. (Obl. System.) Pounds.		Live Load. Pounds.		Total. Pounds.
$c_1\ c_2$	=	+ 8 700	+	8 700	+	28 600	=	+ 46 000
$c_2\ c_3$	=	+ 15 400	+	15 400	+	50 600	=	+ 81 400
$c_3\ c_4$	=	+ 20 200	+	20 200	+	66 000	=	+ 106 400
$c_4\ c_5$	=	+ 23 100	+	23 100	+	74 800	=	+ 121 000
$c_5\ c_6$	=	+ 24 100	+	24 100	+	77 000	=	+ 125 200

		Dead Load. (Vert. System.) Pounds.		Dead Load. (Hor. System.) Pounds.		Live Load. Pounds.		Total. Pounds.
$a_{1\frac{1}{2}}\ a_2$	=	....	—	4 300	—	14 300	=	— 18 600
$a_2\ a_{2\frac{1}{2}}$	=	— 8 700	—	8 700	—	28 600	=	— 46 000
$a_{2\frac{1}{2}}\ a_3$	=	— 8 700	—	12 000	—	38 000	=	— 58 700
$a_3\ a_{3\frac{1}{2}}$	=	— 15 400	—	15 400	—	50 600	=	— 81 400
$a_{3\frac{1}{2}}\ a_4$	=	— 15 400	—	17 800	—	55 000	=	— 88 200
$a_4\ a_{4\frac{1}{2}}$	=	— 20 200	—	20 200	—	66 000	=	— 106 400
$a_{4\frac{1}{2}}\ a_5$	=	— 20 200	—	21 700	—	70 000	=	— 111 900
$a_5\ a_{5\frac{1}{2}}$	=	— 23 100	—	23 100	—	74 800	=	— 121 000
$a_{5\frac{1}{2}}\ a_6$	=	— 23 100	—	23 600	—	76 000	=	— 122 700

+ indicates compression.
— " tension.

NOTE—The following data were used in calculation :

Live Load = 30 000 pounds on two wheels, 6 feet apart.
Dead Load = 400 pounds per lineal foot.
Panel Load, chord, a_1 = 750 pounds.
" " " b_1 = 1 750 "
" " " c_1 = 1 050 "

$a\ c$ = 8′ 0″
$a\ b$ = 4′ 0″

A more elementary method of deducing the strains—although not showing so clearly the action of the different members upon each other—would be, to first resolve the applied loads into the two components of the web systems, and then consider each system as a distinct truss. In following this method, the tensile strains, in the stringer chord, from the oblique system, and the compressive strains in the same chord from the horizontal system, would be found to neutralize each other with the same results as shown above. It must be remembered, however, that the horizontal system can only receive its load at the panel points to which the oblique system is attached.

The stringer has, throughout the description, been termed a "chord," but in reality it is a portion of the web system, and like the other members of that system, receives the heaviest strains in the panels nearest the supporting columns. This will be readily comprehended when it is understood that the direct strain in any panel of the stringer "chord" is due to the diagonals of the same panel, and is equal to their longitudinal component—thus in the panel "b_2 b_1" the direct strain is due to the longitudinal component of the diagonal "b_2 c_1," which produces a thrust from "b_2" toward "b_1"—this thrust is then resisted by the two diagonals of the horizontal system "a_2 $b_{1\frac{1}{2}}$" and "$a_{1\frac{1}{2}}$ b_1." Thus we see that, in each panel, the stringer chord receives its maximum strain when the diagonals of the corresponding panel receive their maximum, and that, in reality, "c b a" is the componential web, while "c" and "a" are the chords proper of the truss.

The horizontal and vertical systems are made of riveted lattice, since they require no adjustment, and their construction of riveted work was considered economical. The horizontal system is provided with counter-diagonals, and was designed to resist all counter-strains produced in it by the live load; the vertical system, however, which carries only dead load, does not provide for counter-strains, and each panel is therefore constructed with but a single diagonal.

The diagonals of the oblique system are square rods with sleeve-nuts, which permit the proper adjustment for camber and rigidity. By this arrangement it is practicable to carry the live load strains through the componential systems, as designed, and allow the vertical system to carry only dead load. The proper adjustment of the rods in the oblique system will relieve any counter-strain in the vertical system which might occur on account of undue deflection under the passing load.

The original conception of the truss contemplated a single diagonal, direct from "b_1" to "a_2," but the angle of inclination between the diagonal and the chord was so oblique, that, in construction, the panel was divided into two, making the inclination of each diagonal about forty-five degrees.

While the treatment of the strains in the three web systems was essentially theoretical, a close examination of the vertical system during a passing load fails to reveal any tendency in the diagonals of that system to resist counter-strains.

The erection of the trusses was not a difficult matter and involved no unusual features. The old girders were removed from their bearings, lowered slightly, and placed on piles of marble. On these girders was constructed a skeleton platform, and on this platform the new trusses were assembled and riveted together. They were then lifted at the ends and swung into position. Messrs. Cofrode & Saylor, of Philadelphia, were the contractors for the construction and erection.

The original design of the trusses provided for a span of 103 feet; but shortly before the contract was awarded, the required width of right of way was reduced by the railroad company and in consequence, two panels of the trusses were taken out, and they were constructed 87 feet 7 inches between centers of columns. The depth of each truss, from out to out of chords of the vertical system, is 8 feet. The width is 4 feet between centers of the chords of the horizontal system. The depth from base of rail to clearance beneath is 2 feet 4½ inches, including a 6-inch wooden stringer. The weight of each iron box girder, of 61 feet span and 2 feet depth, was 28 000 pounds, or 460 pounds per lineal foot; its excessive weight being due to the material required in the flanges to reduce the deflection. The weight of each iron truss, as constructed, of 87 feet 7 inches span, is 34 000 pounds, or 390 pounds per lineal foot.

In measuring the deflection after completion, a piece of twine was stretched tightly from column to column under the stringer chord. There was no wind, and the twine was so tightly stretched that it would quickly come to rest after being disturbed. From this position of rest the position of the stringer chord could be readily measured. The largest stone at convenient disposal, weighing 10 tons (net), was lifted and carried at midspan, half way between the two trusses. The weight of the traveler was also 10 tons, making a total load of 20 tons. The

vertical deflection under this load measured one-half an inch; the horizontal deflection outward measured one-quarter of an inch.

The design of the trusses being novel, their unusual appearance elicited considerable criticism, and during erection prophecies of failure were freely made. It seemed much easier to explain why they must fail than to understand how they could succeed. The light vertical system and the upper chords were the particular objects of criticism, while the stringer itself apparently "had no support." Subsequent developments, however, quieted these apprehensions. The trusses were designed to carry a load of 15 tons on two wheels 6 feet apart, and the first load to which they were subjected was one of the old girders weighing 14 tons. This was carried close to one of the trusses, and with the weight of the traveler was probably fully equal to that for which the trusses were designed. The result of this test relieved all further anxiety as to their efficiency, and the structure has since continued, under constant use, to give entire satisfaction.

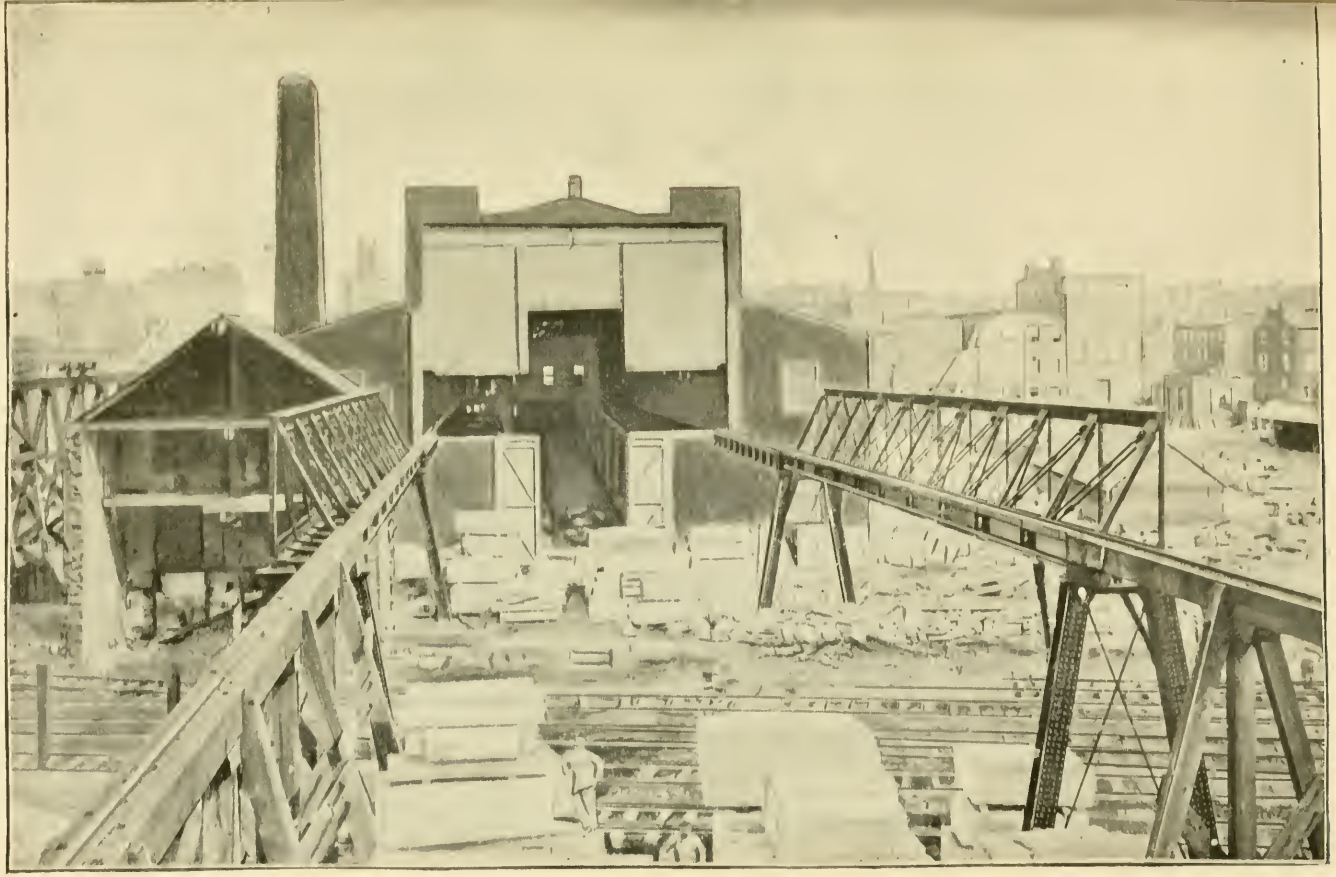


Plate LXXXII. Vol. XXIII, p. 284.

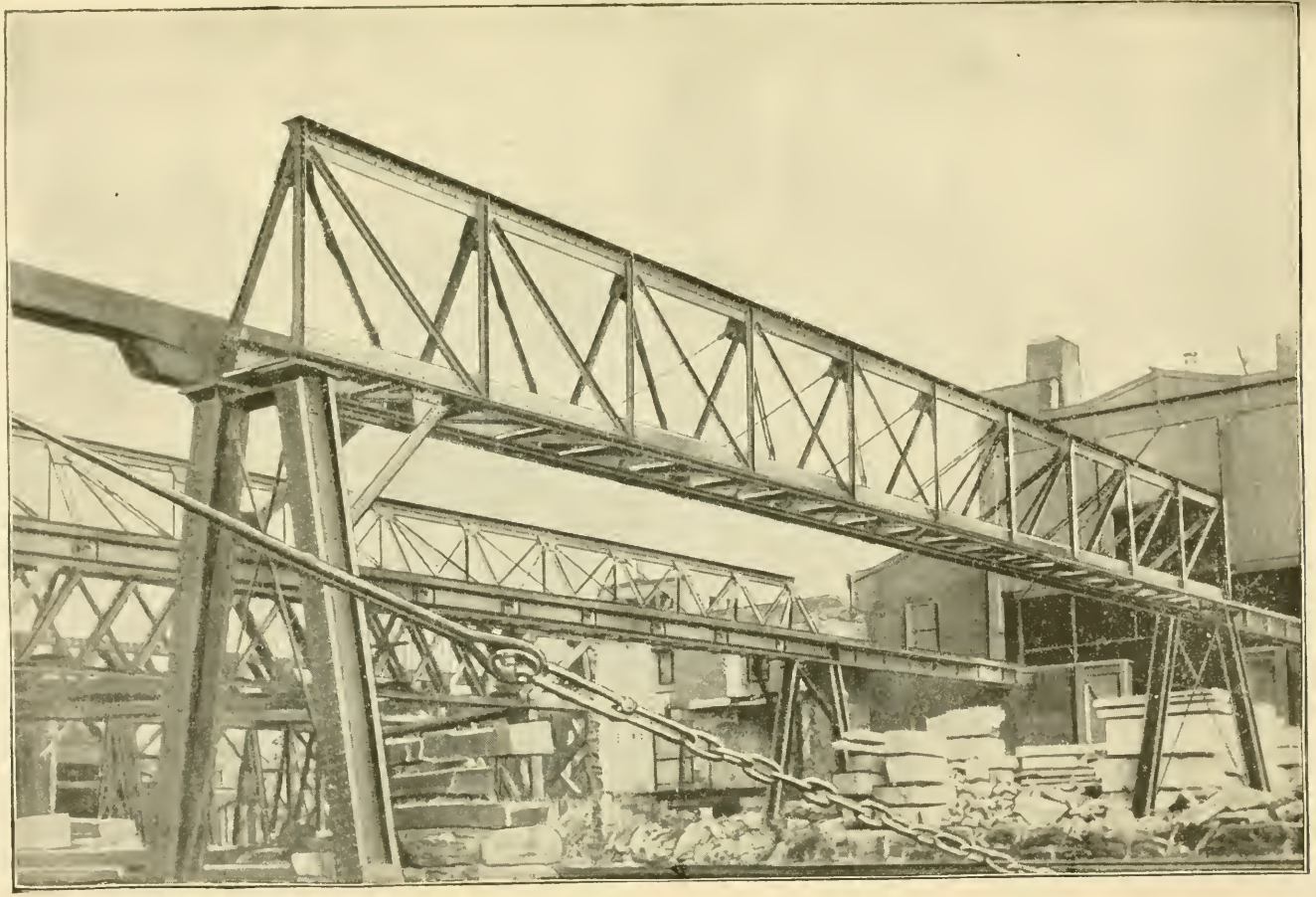
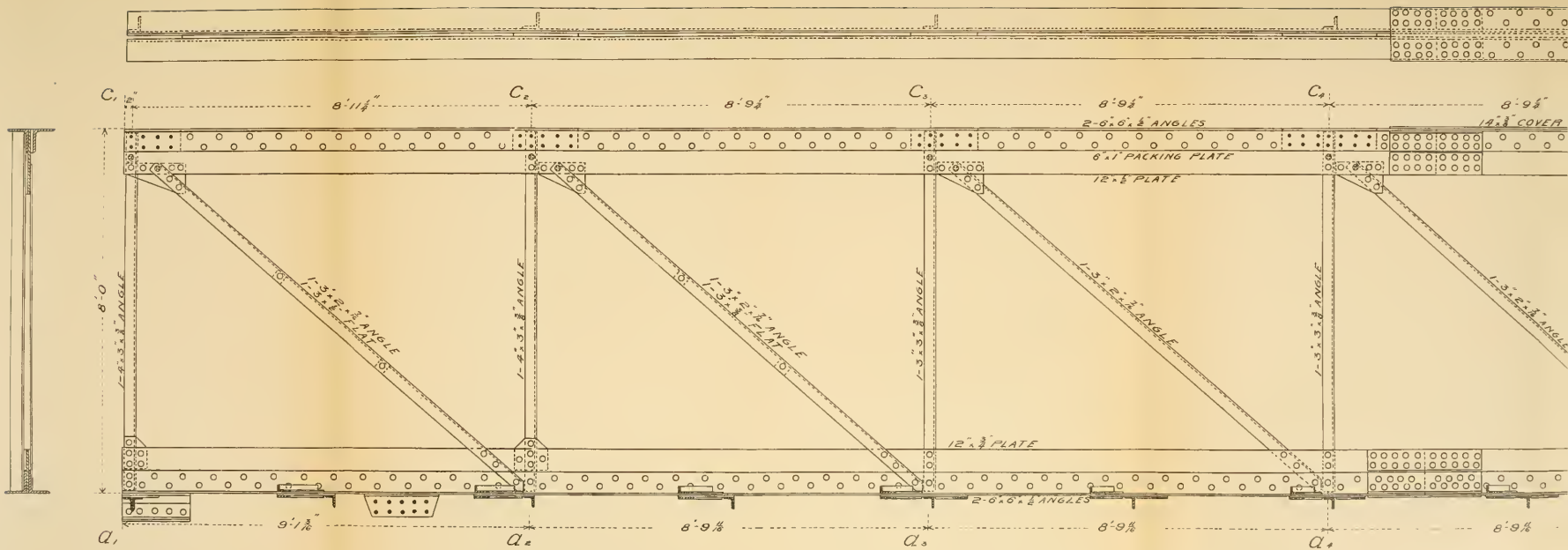


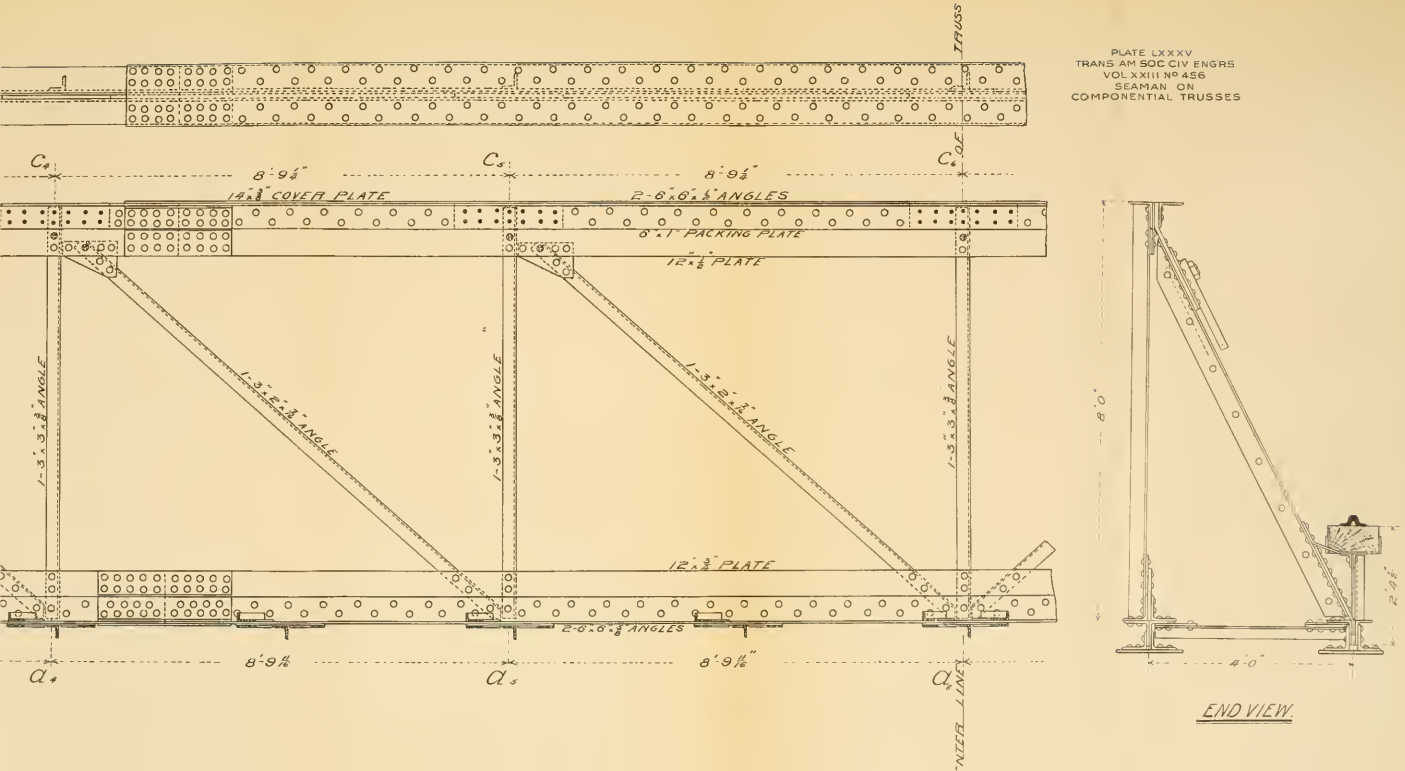
Plate LXXXIII.

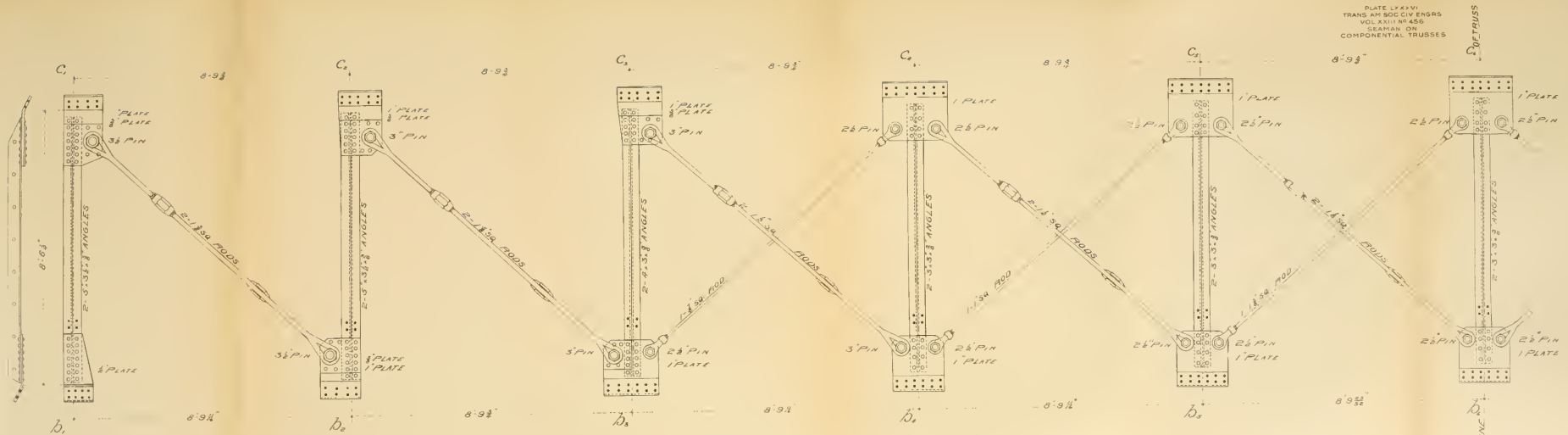
Vol. XXIII, p. 284.



VERTICAL SYSTEM.

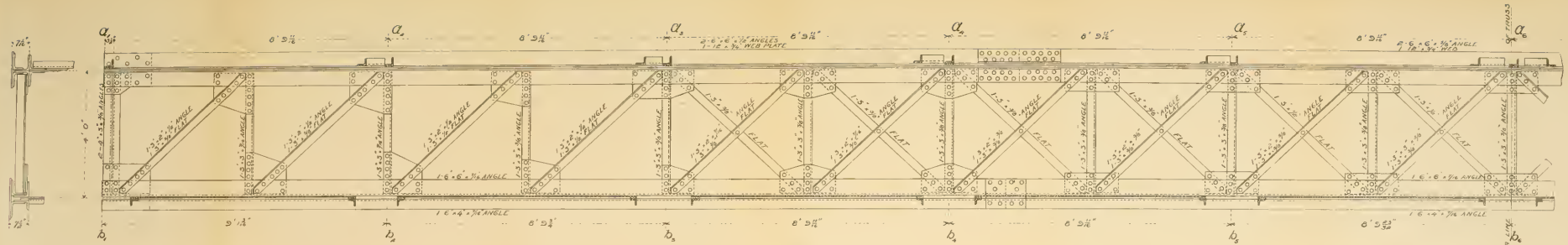
PLATE LXXXV
TRANS AM SOC CIV ENGRS
VOL XXIII No 456
SEAMAN ON
COMPONENTIAL TRUSSES

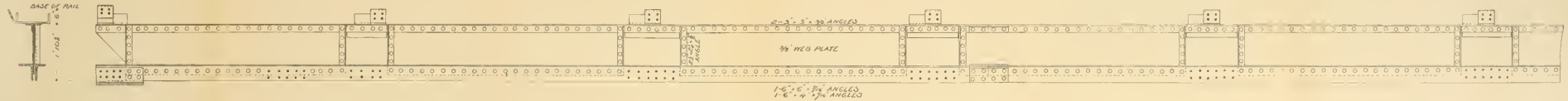




OBLIQUE SYSTEM

PLATE LXXXVII
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXIII. No. 456
SEAMAN ON
COMPONENTIAL TRUSSES





AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXIII.—December, 1890.)

DISCUSSION

ON

RAILWAYS OF MEXICO.*

By MAX E. SCHMIDT, M. Am. Soc. C. E., and
W. BARCLAY PARSONS, M. Am. Soc. C. E.

MAX E. SCHMIDT, M. Am. Soc. C. E.—Referring to the paper by Mr. William B. Parsons, on the Railways of Mexico, the statement appears that 4 per cent. grades are in operation on the Mexican Central Branch from Tampico to Aguas Calientes. This is an error. The ruling grades on this branch are one and one-half (1.5) per cent., except on the two mountain divisions of 27 and 5 mile lengths respectively, where the maximum grades are 3 per cent. All grades are compensated for curvature in conformity with the table below. One and one-half and 3 per cent. express, therefore, the full maximum in operation on this railway.

The statement is also made that 4 per cent. is the maximum grade on the Mexican or Vera Cruz Railway. This is correct only as far as the tangents are concerned. On curves the absence of all compensation virtually increases the maximum to 5 per cent., if not more.

In view of the competition which is likely to take place between the Interoceanic, the Tampico and the Vera Cruz Railways, the latter railway has lately placed on its 4 per cent. non-compensated grades, Fairlie engines weighing over 90 tons. This, in the writer's opinion, is hardly a move in the right direction. As it was impracticable to cut down the grades of this railway, the plan presented itself of placing a center rack-

* Railways of Mexico, by William Barclay Parsons, *Transactions*, Vol. XXII, No. 435, page 233.

rail on the heavy grades of the mountain divisions and operating the same with Abt engines, which at 63 tons service-weight develop a tractive power of 27 500 pounds. The data are not at hand to tell how much the new Fairlie engines will haul on the heavy grades of the Vera Cruz Railway, but their ponderous weight is against them under the best of circumstances. An Abt engine, on a track supplemented by a third rail as stated above, would haul 280 tons, gross train-loads, on 4 per cent. non-compensated grades, and at speeds of 10 miles per hour.

MEXICAN CENTRAL RAILWAY COMPANY (LT.).—TAMPICO DIVISION.—TABLE OF GRADES AND CURVES IN COMBINATION.

Degree of Curve, 25 Meter Chords.		Rate per cent. of Grade.	
		Maximum, 1½ per cent.	Maximum, 3 per cent.
Tangent.		Per cent.	Per cent.
Degrees.	Minutes.	1.50	3.00
1		1.45	2.95
1	30	1.42	2.92
2		1.40	2.90
2	30	1.37	2.87
3		1.35	2.85
3	3	1.32	2.82
4		1.30	2.80
4	30	1.27	2.77
5		1·25	2.75
5	30	1.22	2.72
6		1.20	2.70
6	30	1.17	2.67
7		1.15	2.65
7	30	1.12	2.62
8		1.10	2.60
8	30	1.07	2.57
9		1.05	2.55
9	30	1.02	2.52
10		1.00	2.50
10	30	.97	2.47
11		.95	2.44
11	30	.92	2.41
12		.90	2.38
12	30	.87	2.34
13		.85	2.31
13	30	.82	2.27
14		.80	2.24
14	30	.78	2.20
15		.75	2.17
15	30	....	2.13
16		....	2.09
16	30	....	2.05
17		....	2.01
17	30	....	1.97
18		....	1.93

W. Barclay Parsons, M. Am. Soc. C. E.—In regard to Mr. Schmidt's discussion of my notes on Mexican Railways, I accept with pleasure his correction that the maximum gradient on the Tampico branch of the Mexican Central is 3 and not 4 per cent. I was informed, on what appeared reliable authority, that it was the latter. The point, however, that I wished to emphasize was that the gradient was heavy, and this Mr. Schmidt confirms when he states that there is one stretch of 27 miles of 3 per cent. gradient.

As to whether the curves on the Mexican Railway are compensated, I cannot say; but even if not, the allowance would hardly amount to 1 per cent. or more. The sharpest curve is one of 300 feet radius, I believe, corresponding to about the 15 degrees 30-minute curve of Mr. Schmidt's table. The compensation for this curve, according to his table, is .87 of 1 per cent.

The rate of compensation in the table is substantially 0.04 per cent. per degree of 100-foot chords. It will be noticed, however, that while this rate is uniform for the 1½ per cent. gradient, it is used on the 3 per cent. up to 10-degree curves only, after that the rate is increased.

As to whether it would be economical to have introduced the Abt engine on 4 per cent. gradients, is not by any means clear. In that dry climate the rail is nearly always in good condition, so that the ratio of adhesion is high, and an engine like the Fairlie will give good results. The writer, at present as Chief Engineer, is constructing in the Allegheny Mountains of Northern Pennsylvania a system of lumber railways, using gradients frequently of 3.5 per cent. and at times up to and over 4 per cent. The power used is a Shay engine, a machine with three vertical cylinders driving a horizontal shaft, which is geared to all the wheels, tender included. This shaft is jointed so that the longest rigid wheel-base is 56 inches. Such an engine uses the whole weight for adhesion, and at a ratio of ¼ with a weight of 60 tons, would develop 30 000 pounds, tractive power.

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(Vol. XXIII.—December, 1890.)

THE VENTILATION OF TUNNELS.

By N. W. EAYRS, M. Am. Soc. C. E.

WITH DISCUSSION.

The dictum of the celebrated Dr. Lardner, that steamships could never cross the ocean, for the simple reason that they could not be made to carry fuel enough for the voyage, is well matched by the argument brought forward in the days of the Stephensons, by those who disbelieved in the practicability of railroads, that railways could not come into general use without the necessity of tunnels, and it was manifestly impossible to use locomotives in a tunnel of any considerable length, on account of the vitiated atmosphere which they would produce. Both statements are instances of the dimness of pre-vision to which the most sagacious men are liable.

But those who saw in tunnels an insurmountable obstacle to the introduction of railways had more of reason on their side, for the increasing demands of commerce and travel, which have made possible, and even compelled, the vast improvements in ships and marine engines

since Lardner's time, have so augmented the difficulties in the way of adequate ventilation in railway tunnels that the feasibility of using smokeless engines or some other motor than steam has for some time commanded the attention of railway officials. The question of ventilation presses upon us to-day with increasing force, for it is intimately connected with the question of underground railways, which seem, at present, to furnish the only solution to the problem of rapid transit in those cities where not only the earth beneath, but the heavens above the earth are already occupied with overtaxed systems of transportation.

Another dictum which must be somewhat modified is that of the text-books, which assert that in the case of short tunnels the question of ventilation need not be considered; thus disregarding the effect of the volume of traffic as a factor in the problem. The fact is that the longer tunnels, like the Hoosac, the St. Gothard, the Mont Cenis, and others, seem to be able to take care of themselves in this respect, owing partly to certain physical conditions, but chiefly to their location remote from large cities, so that the volume of traffic through them does not at present, and may not for years, equal the capacity of the tunnel. It is the short tunnels with heavy traffic and trains at short intervals that are most troublesome.

Methods of Ventilation.

Without taking into consideration the expedient seriously suggested in reference to the St. Gothard Tunnel, of supplying the passengers with bags of oxygen, thus making every man his own ventilator, there are, obviously, two methods by which the smoke and gases of combustion may be removed—viz., by natural means and by mechanical appliances. The first of these methods is applicable to all tunnels, whether long or short, provided the volume of traffic does not exceed a certain limit, which limit is determined by the ratio of the necessary intervals between trains to the velocity of the natural current through the tunnel. The existence of this current is dependent upon two causes—one, the difference in temperature of the air inside the tunnel and outside; the other, the difference in barometric pressure between the two portals. These two causes are not operative to any appreciable extent in small tunnels, and ventilation is effected in them either through the action of the winds upon the portals or the mechanical effect of trains passing through the tunnels, or both combined. When a train enters a tunnel it acts as a piston, pushing the air in front of it and imparting a certain velocity

to the air in the rear. From observations made at the St. Gothard this velocity has been determined to be about one-third that of the train itself. This estimate of velocity agrees very well with the writer's experience in the Hoosac Tunnel, where, under natural ventilation and with favorable conditions of the atmosphere, it takes about fifteen minutes for the portion of the tunnel between either portal and the central shaft to clear itself of smoke; the running time of passenger trains being ten minutes between portals, or five minutes between either portal and central shaft.

To secure effective ventilation it is only necessary that the draught through the tunnel, whether natural or artificial, should be sufficient to carry away the smoke and gases in a less time than the interval between the trains; where this is effected the only smoke a train has to contend with is that of its own locomotive.

At the present rate of traffic through the St. Gothard Tunnel a natural current of 1 meter per second clears the tunnel once a day; the average current is 2.61 meters per second. At Mont Cenis a current of 2 meters per second is found sufficient for the full traffic.

Artificial ventilation is effected by means of fans or blowers driven by suitable machinery, the capacity of the fan being based upon the estimated amount of air to be removed in the least interval between trains. In making this estimate it is necessary to take into consideration the section of the tunnel, the character of the walls and the gradients.

As far as the writer is aware, the only tunnels in this country which demand especial attention on the score of ventilation are the Hoosac Tunnel and the St. Louis Tunnel: with both he has been connected as Engineer. They furnish excellent illustrations of the two methods of ventilation; the Hoosac Tunnel being ventilated entirely through the central shaft, and the tunnel at St. Louis being ventilated by means of a fan located near the center of the tunnel. The conditions in the latter tunnel, except, perhaps, as to grades, approach very closely to those that occur in underground railways.

The Hoosac Tunnel.

This tunnel is about 4¾ miles long, and is perfectly straight from portal to portal. The two portals are at the same elevation of 769 feet above the sea level. The grade of the tunnel rises at the rate of 26.4 feet to the mile from either portal to the central shaft. The shaft is

located almost exactly at the middle of the length of the tunnel, and is 1 028 feet deep, measured from the top of the ground to the rail grade. The section of the shaft is an ellipse, the two diameters of which are 27 feet and 15 feet, respectively, the larger diameter being with the axis of the tunnel.

The total amount of brick arching is about one-seventh of the entire length of tunnel. The remainder of the tunnel is in rock. The section of the tunnel within the arching or lining is 24 feet wide and 22 feet high from top of rail to intrados of arch. The rock section is indeterminate, for so much loose rock has been taken down since the tunnel was opened for traffic that the original section has been enlarged several feet in some places.

The portals are provided with doors which are closed in winter to prevent the formation of ice on the sides and roof of the tunnel. The freezing formerly extended in about a quarter of a mile from either portal. Previous to the erection of doors men were employed constantly in breaking down the ice and clearing the tracks. Since 1883 small furnaces have been added, located near the portals, to maintain the temperature above the freezing point. The smoke from these furnaces discharges into the tunnel, but as they consume but about three-quarters of a ton of coke in twenty-four hours they do not affect the atmosphere to any serious degree, and have no effect on the ventilation.

As far as the writer is aware, no observations have been made on the velocity of the air in the tunnel, but it is sufficient to clear the tunnel between the portals and the shaft in about fifteen minutes under ordinary circumstances. On warm, foggy days in autumn and winter the atmosphere in the tunnel is very sluggish, and considerable trouble is experienced from smoke and gas, but even at such times one-half of the tunnel is generally clear. The temperature of the tunnel near the shaft remains pretty constant at about 60 degrees Fahr. all the year round.

The direction of the draught varies with the season, coming in at the portals during the winter season and passing out at the central shaft, while in summer the current is reversed. In winter the draught inwards is so strong that a windlass and double block and fall are necessary to keep the doors from being violently shut and sucked into the tunnel. The writer has seen one set of heavy doors torn from their hinges and carried bodily into the tunnel through the inexperience of a door-tender.

The piston-like effect of moving trains has often been remarked in short, single-track tunnels. It is also quite noticeable in the Hoosac Tunnel. The writer, when standing at the central shaft, has frequently been warned of the entrance of a train at the portal by seeing the flame of his torch change instantly from a vertical position to an inclined one, pointing towards the opposite portal.

When the Hoosac Tunnel was opened for traffic a single track only was laid, and in order to protect trains from the danger of stones becoming dislodged from the sides of the shaft and falling to the track, timber shields or fenders, as shown on the accompanying drawing, Plate LXXXVIII, were erected. These were arranged in such a way as to secure the requisite protection without seriously interfering with the ventilation. When the second track was laid, in 1882, it became necessary to protect that too, and the plan of spiral ventilating galleries shown on Plate LXXXVIII was adopted.

At a point 42 feet from the center of the central shaft, measured in the line of the tunnel, a chamber was excavated in the tunnel-wall, at right angles therewith and extending in about 6 feet. This chamber was carried up the full height of the tunnel, and the floor was placed at the rail grade. From the back of this chamber a spiral gallery, of circular section 15 feet in diameter, was excavated until it entered the central shaft at an elevation of 70 feet above the floor of the tunnel. A similar gallery was excavated in the other tunnel-wall, on the opposite side of the shaft. The two tracks were then covered with a brick arch 3 feet 4 inches thick, completely closing the bottom of the shaft. Over this arch a mass of rubble masonry was laid, 20 feet thick, the courses formed into a rough arch abutting against the sides of the shaft. This gave ample protection to the tracks, and the combined area of the two galleries being larger than that of the shaft, maintained the ventilation of the tunnel as it was before.

It is now eight years since these galleries were constructed, and, although the traffic through the Hoosac Tunnel has largely increased during that interval, there has been noticed no injurious effect on the ventilation. When the plan was under consideration in 1882 it was a serious question whether the abrupt turn at right angles and the spiral tubes would not prove such an obstacle to the air-current as to affect the ventilation.

But, however satisfactory the ventilation of the Hoosac Tunnel by

natural draught has been hitherto, the time is fast approaching when it will cease to be sufficient and recourse must be had to artificial ventilation or some other motor than the ordinary locomotive. The writer was recently informed by the Chief Engineer of the Fitchburg Railroad, Mr. E. K. Turner, that the greatest number of trains in one day passing through the tunnel is sixty-five, "which is very near the limit which can be safely run under present conditions." He is strongly of the opinion that, "with the increasing number of trains passing through the tunnel, it will soon become necessary to make some changes either in ventilating or keeping smoke and gas out."

During the year 1889, the total number of freight cars passing through the tunnel was 254 112; of passenger cars, 18 600. Average number of freight cars to a train, thirty. These figures do not represent the total movement of trains and engines in the tunnel, for they do not include work trains, light engines, or the pushers that run from North Adams to central shaft and back.

The St. Louis Tunnel.

Turning now to an illustration of the other method of ventilation, that by mechanical means, a brief description of the St. Louis Tunnel will be in order. This tunnel forms the connection between the St. Louis Bridge and the Union Depot at 12th and Poplar Streets in the city of St. Louis. The tunnel proper commences at 3d Street, passes under Washington Avenue as far as 7th Street, where it curves to the south, under the block and St. Charles Street, to 8th Street, and then runs in a straight line under 8th Street to an open cut at Clark Avenue. The tunnel is 4 095 feet in length between portals, and is really a double tunnel, a central wall separating the two tracks throughout, except at the fan and at the Custom House, between Locust and Olive Streets. This central wall has openings in it at intervals of 25 feet, for places of refuge for trackmen and for passing from one track to the other. As will be seen on the accompanying plan and profile, Plate LXXXIX, the line of the tunnel has several curves, the total amount of curvature being 95½ degrees; the largest portion of this curvature is in a length of 740 feet of 11½ degree curve. The heaviest grade, 70 feet per mile, is at the eastern end.

When the tunnel was completed, openings through the roof were left in Washington Avenue, St. Charles Street and 8th Street, communicat-

ing directly with the open air. It was expected that these openings would keep the tunnel free, without any other means of ventilation, but it was found that the increasing number of trains kept the tunnel filled with smoke and gas, and loud complaints were made because of the volume of smoke which poured out into the streets through the openings. Coke was used as fuel in the locomotives for more than a year, but while the quantity of smoke emitted was decreased, the volume of gas was not diminished. The hope of adequate ventilation by natural draught was abandoned, and the late C. Shaler Smith, M. Am. Soc. C. E., then Consulting Engineer of the St. Louis Bridge Company, was called upon to propose a remedy. For several months Mr. Smith experimented with models of various kinds of fans, air screws and blowers, and many observations were made on the flow of air-currents in the tunnel and the friction of the air against the sides and roof. Unfortunately, his notes and memoranda are not now in existence. The models were all constructed to the same scale, and that form of machine was adopted which gave the best experimental results. The service fan was then constructed on the lines of the model which had been selected.

Plate LXXXIX shows the arrangement of the fan-house and sections of the fan. The fan-house is located on St. Charles Street, near the middle of the length of the tunnel.

The fan consists of a pair of steel cones placed butt to butt and riveted together. These cones carry the blades, sixteen in number, 3 feet wide. The fan itself is 15 feet in diameter and 9 feet wide. The following proportions were used in constructing the fan. Considering the area of inlet as unity, the area of issues was made 0.42, and the area of discharge at the top of the stack was 0.61.

The stack is made of boiler iron, 37 feet diameter at the base and 15 feet diameter at the top. The height is 130 feet.

The fan is run by a Herreshoff compound double-cylinder non-condensing engine, of a maximum capacity of 190 horse-power. The speed varies from 70 revolutions per minute during the middle of the day, when passenger traffic is light, to 110 revolutions per minute during those portions of the day when the tunnel is practically given up to passenger trains. This is from about 6.40 to 9 A.M. and from 4.30 to 8.30 P.M. At 110 revolutions the periphery speed is 5 180 feet per minute; at this speed the tunnel is cleared of smoke in four and one-half minutes from the time the locomotive passes the fan, going east,

and in three and one-half minutes from the time a west-bound train passes the fan.

It has been estimated that the total amount of air to be removed in order to clear the tunnel of smoke from any one train is 2 750 000 cubic feet. To get the best results from the artificial ventilation it was found necessary to close all other ventilating openings.

The writer recently made some observations with a Biram anemometer on the amount of air issuing from the mouth of the fan. The anemometer had been tested at the Washington University in St. Louis, some years ago, and as it had not been used since, it was assumed to be correct. The method of making the observations was as follows: The area of issue—9 feet square—was divided into nine equal parts, and several readings of the anemometer were taken in each small division, with the fan revolving at 70 revolutions and at 110 revolutions per minute, the minimum and maximum speeds. All defective observations, owing to the presence of trains in the tunnel, were thrown out. The mean of all observations taken in the small divisions was computed, and the mean of these means was taken as the amount of air passing through the area of issue in one minute. It was found that at 70 revolutions the amount of air discharged was approximately 200 000 cubic feet per minute, and at 110 revolutions 240 000 cubic feet per minute. Without having at hand the notes of Mr. C. Shaler Smith, the writer is at a loss to understand the discrepancy between Mr. Smith's results and his own. Mr. Smith stated that the total amount of air to be removed from the tunnel was 2 750 000 cubic feet, and that this was removed in six minutes —four and one-half minutes from the time the train passed the fan. This would give a capacity of 450 000 cubic feet per minute. But the great fans at the Mersey Tunnel, 40 feet in diameter and 12 feet wide, running at 45 revolutions per minute, are rated at 200 000 cubic feet per minute, and the total of all four fans—two 40 feet in diameter and two 30 feet in diameter—is but 500 000 cubic feet per minute.

Up to within two or three years the ventilation of the St. Louis Tunnel has been considered fairly good. In certain states of the atmosphere, however, as in the case of all tunnels, the smoke moves very sluggishly, and in a portion of the tunnel the air is very foul. With a strong westerly wind, for instance, the smoke collects at the eastern half of the tunnel, and the fan, while not powerful enough to draw the smoke to the ventilating stack against the current of air entering the

western portal, is yet sufficient to keep the body of smoke from drifting out of the eastern portal.

It is only a question of a few years, however, before more adequate means of ventilation will have to be provided, owing to the rapid increase of traffic. During the year 1888, the total number of trains and light engines passing through the tunnel was 61 174, an average of 167 trains for every day in the year, or about eight and one-half minutes between trains. The time-card in effect June 15th, 1890, shows thirty-eight regular passenger trains between 6.40 A.M. and 9.10 A.M., or one train every four minutes in the tunnel.

European Tunnels.

In closing this paper a brief glance will be given to the conditions of some of the principal European tunnels as regards ventilation.

In the mountain tunnels of the continent of Europe natural ventilation has been found, so far, to be sufficient. This subject received a great deal of attention at the opening of the Mont Cenis and of the St. Gothard, and a thorough investigation of the meteorological conditions existing at these tunnels has been made.

At St. Gothard Tunnel the natural current of air was found to be due to two causes: difference of temperature between the air inside and outside the tunnel, and the difference in height of barometer at the two portals, which differ in elevation by 36 meters. This difference in barometric height is always sufficient to produce a definite current from north to south (the north portal being the lower) when the temperature of the air at the two portals is nearly or quite the same. The air current from the north is pretty constant during the summer months. In general the tunnel is so clear that either end can be seen from the middle. The traffic, however, of the Alpine tunnels is yet comparatively light.

The artificial ventilation of tunnels has been carried to a greater extent in England than elsewhere. The Mersey Tunnel is the most noticeable example of successful ventilation under heavy traffic and with heavy grades. This tunnel is 4 960 feet long, under the river, and is 26 feet wide; it is double-tracked throughout. The grade in a portion of the tunnel is 196 feet to the mile.

The principle on which the ventilation was planned was to admit fresh air at the stations and draw it either way to points midway between stations, where the ventilating fans were placed. An auxiliary

tunnel or air drift 7 feet 2 inches in diameter runs parallel with the main tunnel, and is connected with it and the stations by sliding doors, so that air can be drawn from any point desired. The fans are four in number, two 40 feet diameter and 12 feet wide, and two 30 feet diameter and 10 feet wide. Their collective capacity is 500 000 feet per minute; average number of revolutions per minute is forty-five.

For purposes of ventilation the tunnel is divided into four sections, one fan being allotted to each ; but by means of the doors in the air passages the fans can be made to do each other's work, and no complete stoppage of ventilation is possible. The 30-foot fan at Liverpool ventilates the James Street station and the section of the tunnel between that station and the terminus. The capacity of this fan is about 120 000 cubic feet per minute. The 40-foot fan at Liverpool ventilates the section of the tunnel between James Street station and the center of the river. Capacity about 130 000 cubic feet per minute. The 40-foot fan at Shore Road, Birkenhead, ventilates the section between the middle of the river and the Hamilton Square station, and has a capacity of 130 000 cubic feet per minute. The 30-foot fan, the fourth in the series, is located at Hamilton Street, nearly midway between Hamilton Square station and Borough Road, and has a capacity of about 120 000 cubic feet. The combined fans have a capacity about one-seventh that of the entire tunnel. These fans are all built on the lines of the well-known Guibal fan.

About three hundred trains a day pass through the tunnel, giving a maximum train service of one train each way every five minutes. With this heavy traffic, and with the severe grades, the ventilation of both tunnel and stations is excellent.

In the opinion of most engineers who are interested in the subject, the hope of successful ventilation lies in the possibility of keeping the smoke and gas out of tunnels rather than in removing the vitiated atmosphere. For underground railways, where, though trains may be heavy, grades will probably be light, electricity offers good ground for hope of a solution of this problem. Fortunately, in mountain regions where grades are liable to be severe, the traffic will be comparatively light for many years to come, and no special means of ventilation will be necessary. But even in these tunnels, such as the Arlberg and the Simplon, the possibility of using smokeless engines or a method of suspended firing has been under consideration.

DISCUSSION.

Augustus W. Locke, M. Am. Soc. C. E.—I fully agree with Mr. Eayrs that the true method of maintaining a good atmosphere in tunnels is to move the train by some motive power other than the ordinary locomotive, which shall not fill the whole tunnel with smoke and steam. The smoke and gas render the atmosphere unfit for breathing, and in long tunnels laborers and trainmen are made sick thereby. The smoke and steam cut off the sight and sound, so that trains cannot be seen or heard at any long distance, and visual signals are not reliable.

So far as I have observed, the motion of trains has very little beneficial effect on the ventilation. A train often changes the current, but it changes back to its original direction as soon as the train leaves the tunnel. If the train and the current are both going the same way, then the train accelerates the current somewhat, but not over 3 or 4 miles an hour. But if the current is traveling west, for instance, and a train enters going east, the current is either reversed or else brought to a stand-still. If reversed, the smoke which had previously been traveling toward daylight is driven back and has to go the same ground over again. So the good that one train does is offset by another running in the opposite direction.

Steam is as great an obstruction to the sight as smoke, although it does not seriously injure the air for breathing, but rather improves it.

The passage of two or three freight trains going up grade at short intervals produces a dense atmosphere, so that a large torch cannot be seen over 30 feet. Then if there is not much draught and the air continues nearly stationary, those who breathe it will soon begin to have headache and perhaps dizziness, and sometimes they have been made unfit for duty, and have fallen down and have been carried out on push cars. Much time is unavoidably lost by workmen in long tunnels, because they cannot see to work, and much more on account of the depressing effect of the bad atmosphere.

Smokeless engines would not be a remedy, as the steam and the products of combustion would still be there.

A tunnel supplied with motive power other than the steam locomotive would be useful to the same extent as the same number of miles of track outside. Now, long, smoky tunnels have to be run with unusual care. Ordinary train flagmen are easily bewildered in a smoky tunnel, and are not to be relied on. Neither can any kind of a visual signal be depended on. Torpedoes or other audible signals have to be used. The absolute block is the only safe reliance.

The time seems to have arrived when tunnels with a heavy traffic need to be kept clear of all smoke, gas and steam, and be lighted by electricity.

At the Hoosac Tunnel, with which the writer was connected as an engineer for eighteen years, during twelve of which he had it under his care, the ventilation has always been bad for many days out of each year. Sickness among workmen happens occasionally, and whole gangs have been taken sick there.

It is not probable that fans can be used to advantage in this tunnel on account of its large area and the long distance the air has to travel. Any system of artificial ventilation to be of any service ought to clear the tunnel in twenty minutes, which is about the best that natural ventilation accomplishes in the coldest weather. The expense of operating the necessary machinery to move so large a volume of air in so short a time, to say nothing of the expense of preparing an underground chamber to receive the machinery, makes it very doubtful whether that method can ever come into use there. And even if it were tried it would by no means insure a clear atmosphere. Every freight train would, as now, fill half the tunnel with gas, smoke and steam.

The grade at the Hoosac Tunnel is 59.94 feet higher at the central shaft than at the portals. The total amount of brickwork is 7 573 lineal feet, a little less than a third of the whole. The tunnel is 26 feet wide where lined, and 24 feet wide where unlined. The loose rock which has been removed from the roof and sides in the rock section has no visible effect upon the size of the tunnel, but has rendered it from year to year more secure.

The temporary timber shields, as well as the masonry and galleries at the bottom of the central shaft, were designed and constructed by the writer, and they were not intended to have any effect upon the ventilation worth speaking of, but were solely for the purpose of protecting the tunnel from falling stones, leaving sufficient openings for air, smoke and steam to travel freely. The air galleries were planned with ample area and easy curves, as shown on the writer's original plan of April 15th, 1880, a copy of which accompanies Mr. Eayrs' paper. It was not intended that the current up or down the shaft should be at all retarded on account of the construction of the galleries, and no trouble of that kind has ever arisen.

The timber shields were only a temporary expedient, and proved to be a poor reliance, although constructed of 8 x 14-inch bridge timber very strongly bolted to the sides of the shaft. Before starting to construct the work at the bottom of the shaft two miners were lowered down about 100 feet from the top to examine some loose rock which was known to be hanging there. They found one dangerous piece, weighing upwards of 500 pounds, and let it down. It struck the shield at the bottom (about 1 000 feet below) and completely demolished it.

Lewis M. Haupt, M. Am. Soc. C. E.—The subject which is brought to our attention in this carefully prepared paper is one of great importance in view of the demand existing in our overgrown communities for rapid transit.

The author has very clearly drawn the line between long and short tunnels, and has shown why the *bête noir* of the former is of comparatively little moment, while in the latter it becomes of serious consequence, due to the frequency of train movements. Probably nowhere is this more apparent than in the Bergen Hill Tunnel, less than 1 mile long, on the Central Railroad of New Jersey, where long trains follow each other with a headway of only a few minutes, and where the air is so dense with gas and smoke that the engine-drivers must run by faith and not by sight. Although this has been a serious matter for some years back, when a longitudinal brattice work was contemplated, I have not learned that anything has been done to afford relief.

Long before the Hoosac Tunnel was demonstrated to be even practicable, the question of its ventilation assumed great importance, and it was strongly urged by its opponents that it would be impossible to breathe in so contracted a space. Later, when it was proposed to build the central shaft, the matter of its effect upon the tunnel as designed, namely, for single track, demanded careful consideration. The position taken by H. Haupt, Chief Engineer and contractor, was that for a single-track tunnel of this length the piston-like action produced by the passage of the trains would be sufficient to change the air in the tube, whereas if the central shaft were built, the foul air and smoke at the crown would rise in the shaft as the train approached the summit and be sucked down again after its passage, leaving a cloud of vitiated air in the center which it would be difficult to remove. The ventilation, it was predicted, would be greatly assisted by the physical conditions at the site, for the portals face east and west, with two summits between, causing considerable differences of temperature and pressure at the ends which would favor the circulation.

This, however, would be neutralized to a great extent by the winter doors mentioned in the paper, which must also interfere with the ventilation by the air shaft at the apex of grade. It was further proposed when the expense should be warranted by the traffic to open a number of lateral drifts and construct a twin tunnel beside the first, and on its completion to close the ports used for construction and thus secure the benefit of the mechanical ventilation due to train movements for a double-track tunnel with a rock brattice work at less expense both for first cost and maintenance. But the policy of the State changed with its administration, and an engineering work which would have been completed with profit for $2 000 000, became a political machine which ultimately cost nearly $17 000 000.

The necessity of winter doors in that latitude soon became apparent, for during construction large stalactites and stalagmites of ice were frequently struck by the construction trains in winter.

The statistics of the work are well stated by the author, and it may be interesting to add that in September, 1888, the Chief Engineer, Mr.

E. K. Turner, reported to me the number of trains per diem as forty, of which eighteen were passenger. If it is now sixty-five, it indicates an increase of 60 per cent. in two years, and impresses the importance of early measures for the improvement of the ventilation.

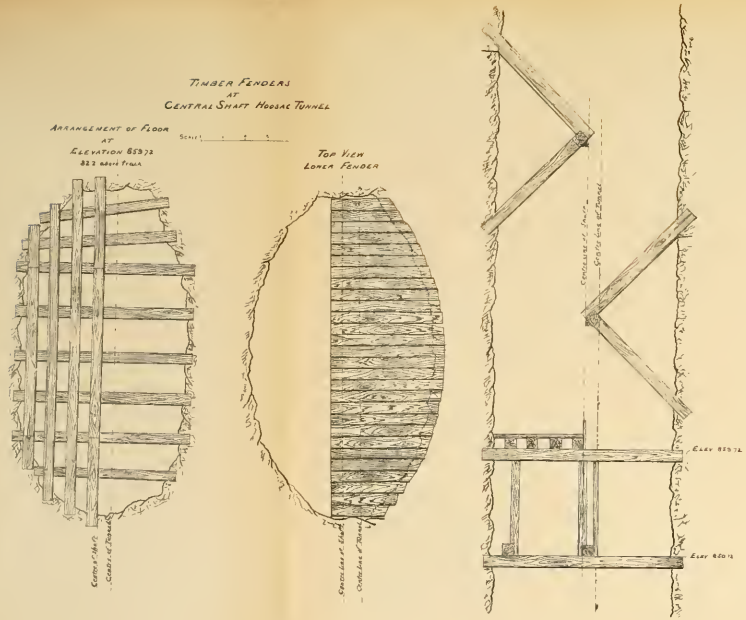
The author has already cited the celebrated Mersey Tunnel, with its steep grades of 1 in 27, heavy traffic and soft-coal locomotives, and I only refer again to it to confirm the statement as to the purity of the air, which is much better than that of the Metropolitan Underground Railway of London. In Glasgow I saw in use a device for assisting engines up steep inclines through the tunnel, which eliminated the use of steam entirely. It was merely an endless cable to which the engine was attached while in motion. The problem of rapid cable propulsion with the object of dispensing with locomotives for underground traffic has, doubtless, received its highest present development at the hands of the Union Traction Cable Company of New York; but where this method is not practicable it has occurred to the writer that a number of small fans at frequent intervals, operated by electric motors, would prove simple and efficient where cheap power is available, and where the usual railway rolling stock must be used in the tunnel, at least for through travel.

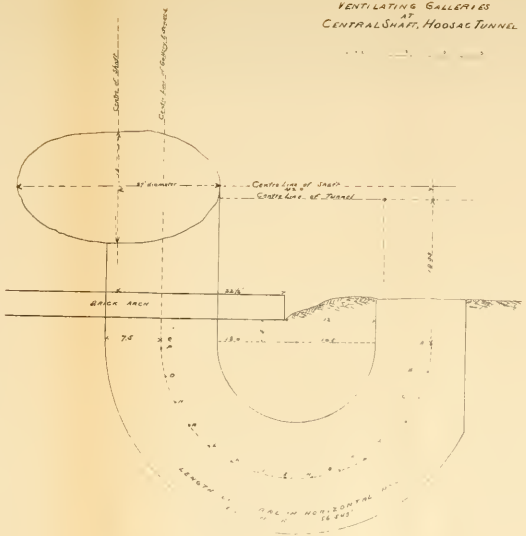
N. W. Eayrs, M. Am. Soc. C. E.—The phenomenon of a column of smoke rising and falling in a ventilating shaft or stack, which was anticipated by Mr. Herman Haupt with reference to the central shaft of the Hoosac Tunnel, is very noticeable in the ventilator stack of the St. Louis Tunnel. When the fan has been stopped for some time the stack becomes filled to the top with smoke, which remains stationary until a train enters the tunnel. As the train approaches the fan, the smoke column rises, and continues to rise until the train passes the bottom of the shaft, when the motion is reversed, and a strong current of cold air is drawn downward from the top of the stack into the tunnel by the suction produced by the receding train. This action of the moving train is, however, never sufficient to clear the stack.

I quite agree with Professor Haupt that better ventilation than now exists would probably be obtained by the use of a number of small independent fans distributed at suitable intervals along the tunnel. The object aimed at is, of course, to empty the tunnel of vitiated air in the shortest possible time, and this is more readily accomplished by taking the air from several points than by moving the whole mass of air from the portals to a central ventilating station. I think, however, that the same result would be accomplished with less expenditure for operation and maintenance by making use of an auxiliary ventilating tunnel communicating with a central ventilator station.

It is interesting to learn that the idea of a ventilating tunnel, as successfully applied at the Mersey Tunnel, was contained in the original designs of the Hoosac Tunnel.

But after all that has been accomplished in the matter of ventilation of tunnels or underground railways, it must be acknowledged that a satisfactory solution of the problem has not yet been reached. Even with the Mersey Tunnel, which is, without doubt, the most successful instance of artificial ventilation, it must be only a question of time when the volume of traffic will overtake the capacity of the ventilating apparatus. The complete solution of the problem seems to be in the adoption of some other motor than steam. As yet none of the expedients suggested meet the requirements fully, but electricity, at least for underground city traffic, where the trains, though frequent, are short, and where, from the nature of the traffic, the need of absolute freedom from smoke and gas is most keenly felt, seems to offer the most promising field.


TIMBER FENDERS
AT
CENTRAL SHAFT HOOSAC TUNNEL
ARRANGEMENT OF FLOOR
AT
ELEVATION 85372
TOP VIEW
LOWER FENDER
ELEV 85372
ELEV 85072


VENTILATING GALLERIES
AT
CENTRAL SHAFT, HOOSAC TUNNEL
Centre of Shaft
Centre Line of Shaft
Centre Line of Tunnel
BRICK ARCH
LENGTH OF GALLERY IN HORIZONTAL PLANE

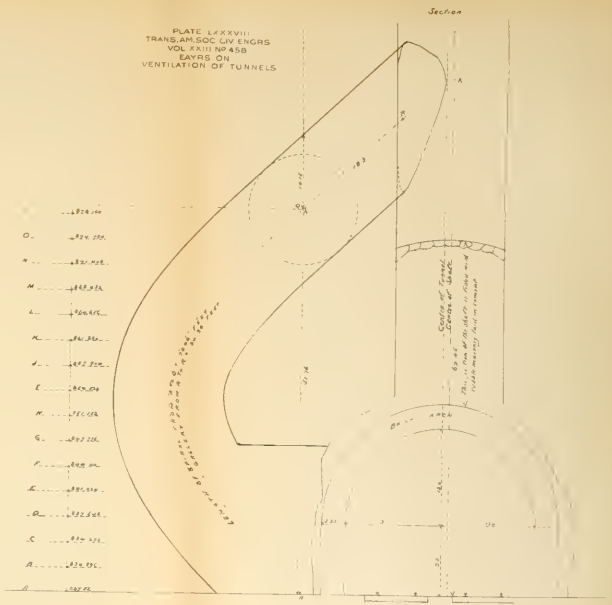


PLATE LXXXVIII
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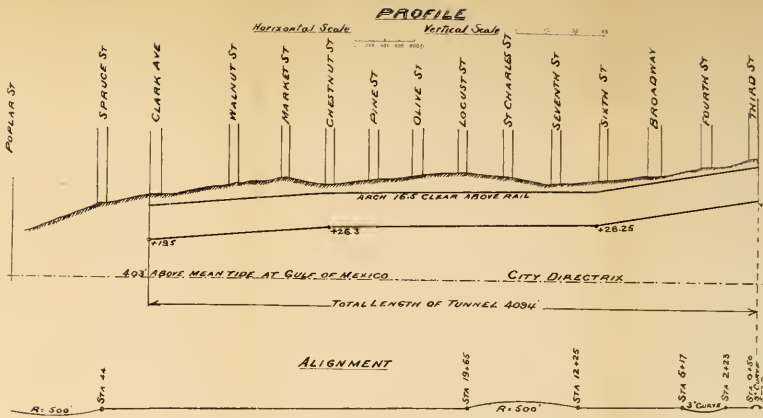
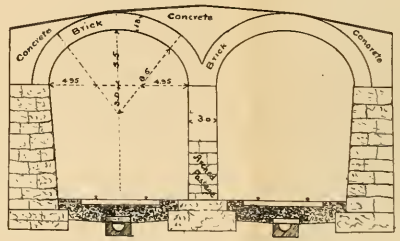
VENTILATION OF TUNNELS
Section
BRICK ARCH

Location of St Louis Tunnel





SECTION:

ST. LOUIS TUNNEL

PROFILE ALIGNMENT AND SECTION

PLATE LXXXIX
TRANS. AM. SOC. CIV. ENGRS
VOL. XXIII No. 458
EAYRS ON
VENTILATION OF TUNNELS

VENTILATING FAN AND CHIMNEY OF ST. LOUIS TUNNEL.

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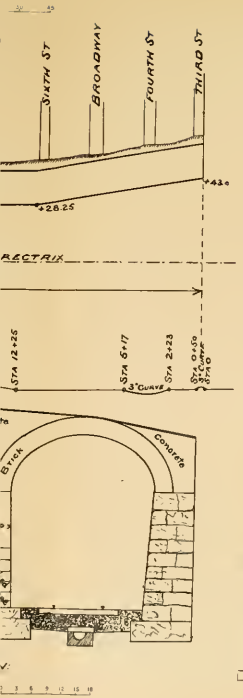
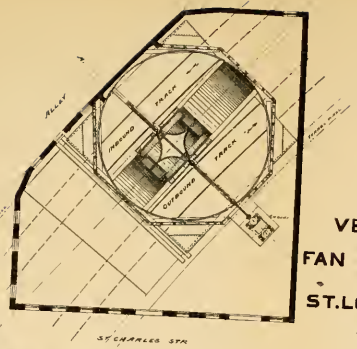
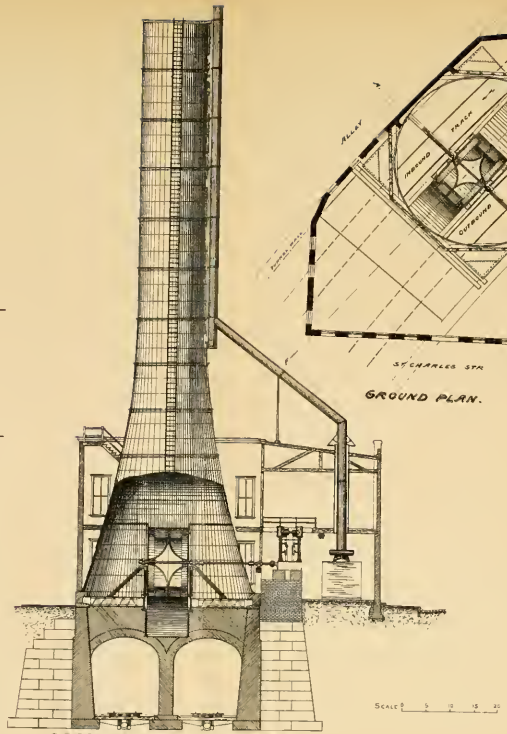
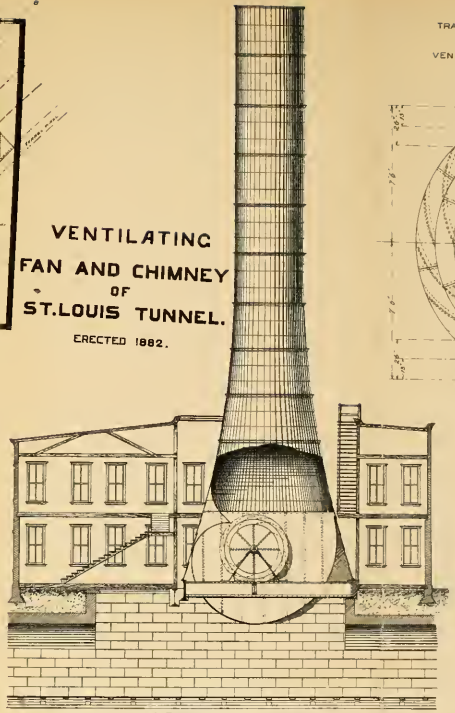




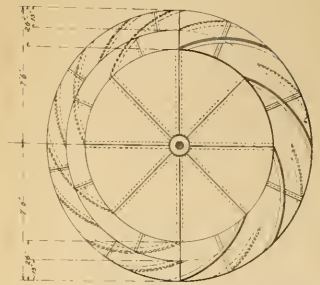
GROUND PLAN.



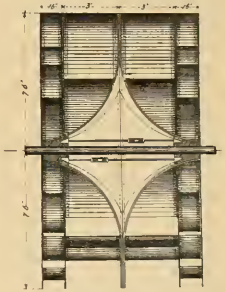
CROSS SECTION ON LINE A.B.



SECTION ON LINE F.G.



SIDE ELEVATION OF FAN.



SECTION THROUGH FAN.

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(Vol. XXIII.—December, 1890.)

THE CAUSE OF RAIN AND THE STRUCTURE OF THE ATMOSPHERE.

By FRANZ A. VELSCHOW.

In a paper entitled "On the Cause of Trade-Winds,"* which I recently had the honor of reading before the Society, I gave my reasons for assuming that the actuality which lies behind the really abstract terms "a center of high pressure" or "an anti-cyclone" is a body of unsaturated or dry surface-air, or what may be called an air-cushion. I now propose to continue this train of thought by dealing in a similar way with low pressures, or cyclones, thereby trying, if possible, to arrive at a definite conclusion as to the actual cause of rain; rain being the most prominent feature of cyclones, or low pressures.

The difficulty in approaching this subject lies perhaps herein, that, as Mr. Scott says in his "Elementary Meteorology," 1887, "almost every one imagines himself a born meteorologist," and therefore in all likeli-

* *Transactions*, Vol. XXIII, August, 1890.

hood, if this be true, almost every one of my present audience has formed for himself a more or less definite opinion of the cause of such an every-day occurrence as rain. To shake this faith a little, and to show you that we here really stand before a problem which has not as yet been quite solved, I may commence by quoting what a man of Mr. Scott's experience says. "We must admit," says he, "that the study of weather has made next to no progress at all in gaining an insight into the agencies which are at work in producing the various phases of weather;" and, "unless this be secured by careful and long-continued attention to a few simple and obvious principles, the labor bestowed on the most complete mathematical discussion of the results will be thrown away."

It is indeed a curious fact that the more pains meteorologists have of late years been taking in trying to bring the accumulating facts of observations to agree with theory, the farther they seem to have gotten away from their goal. They may not all admit this, but it is a sign of a wise man that he admits when he knows nothing; and, as we have just seen, Mr. Scott for one is evidently fully aware of the deficiencies of his science, which he declares "can hardly be called a science as yet."

To make you a little familiar with the difficulties we have to encounter, I may commence with a brief examination of the rain theories which at present are most in vogue with meteorologists. These have been condensed in the following words by Mr. Scott:

"Rain is produced by the chilling of air more or less charged with moisture. This is effected in various ways, of which the following are the principal: 1. The ascent of a current of damp air, which is chilled as it rises; 2. The contact of warm and damp air with the colder surface of the ground, as in case of our own west coasts in winter (England), where the land is colder than the sea-surface; 3. The mixture of masses of hot and cold air."

Of these three causes, No. 3 is by the author himself placed *hors de combat*, when he states that Dr. Hann, of Vienna, has calculated, "that even by assuming a very extreme case, which could hardly occur in nature, there could not be produced as much as the twentieth part of an inch of rain."

Cause No. 2 is by the author partly included in Cause No. 1, as the sloping land-surface causes the air moving against it to ascend. As to the other part of it, I fail to see how contact between a cold surface and

warm air can produce rain. It can produce deposit of dew, as, for instance, when we bring a glass of cold water into a heated room; but rain always falls from a considerable distance from the ground, and is therefore not created at the place of contact of the air with the land-surface.

We are therefore now reduced to Cause No. 1 as the only possible one among these causes of rain. This is, however, worse than no cause at all, as may be seen from the following simple experiment: If, under the piston of a strong glass cylinder, we have air saturated with moisture, and press the piston down, a portion of the moisture is condensed into water, as is seen by the mist formed, and the trickling of dew down the inner surface of the glass vessel. The temperature is raised by the compression, but not sufficiently to prevent condensation from taking place. If we now draw the piston back to its first position, we find the air under the piston in the same condition as when we started the experiment. But this means, that, by expanding the air, the moisture which was condensed into water by the compression has again evaporated. The air, therefore, gets chilled by the expansion, but not sufficiently to prevent this evaporation from taking place. (See Note I.) By drawing the piston suddenly out a mist or cloud is often momentarily formed in the vessel. Engineers will often have noticed the same phenomenon when the compressed air in a diving-bell is suddenly let out. As, however, any sudden or violent change in pressure never occurs in nature or in the open atmosphere, this phenomenon can have no connection whatsoever with the cause of rain.

The consequence is, that the chilling produced by expansion during the ascent of a current of damp air can under no circumstances be the general cause of rain. The experiment, however, shows that condensation or rain can be produced by a body of saturated air being brought under greater pressure; and of this we will just make a passing note.

The ascent of a current of damp air is by most meteorologists considered to be the chief cause of rain, and is supposed to take place at the center of a cyclone. It is thus maintained that there is a certain inward movement of the circulating surface air in a cyclone, and that for the air (this is supposed to be always damp air) which is carried by it toward the center there is no other means of escape but to rise at the center. How absurd this explanation must appear to anybody who has been living in deserts or arid districts will be observed when I mention that, while I was in Australia during a period of very severe drought, a

break in the drought was caused by a series of cyclones crossing the country,* entering in the northern part of New South Wales, and passing out again through Victoria, thereby drenching a narrow strip of land about 50 miles wide and 400 or 500 miles long with rain, while on both sides the drought continued uninterruptedly. It seems difficult to explain how this cyclone should have gathered its supply of moisture from moisture rising from the dry surface air over a perfectly dried-up country.

Numerous equally striking instances of the hollowness of the rising-current cyclone-theory might be mentioned; *e. g.*, in a place like New York, where the difference in temperature between summer and winter is often as much as 80 degrees Fahr., the contents of moisture in saturated surface air is 20 times greater during summer than during winter, and for two equally conditioned cyclones, that is, equal with regard to distribution of pressure and force of wind, there would rise 20 times more moisture at the center during summer than during winter, and we might therefore expect a much heavier downpour from the summer cyclone than from the winter cyclone, but no such difference is observed ; the downpour per hour is practically the same.

What we want is evidently a rain theory which is capable of accounting for rain, whether the surface over which a cyclone passes is wet or dry, or whether it is giving off vapors or not; and I have taken pains to show how utterly incapable the existing theories are in this respect, so as to clear the atmosphere from old cobwebs which might stand in the way of an entirely different view of the whole question, being well aware of the opposition with which new theories are generally met at the beginning.

In my pamphlet on "Drought" I called attention to the aqueous vapor as the element of the atmosphere to which some unknown quality was likely to adhere, and by knowing which we should be able to explain the whole atmospheric puzzle. With the object in view of finding this secret, I undertook, in April, 1888, a series of observations from the tower of the Rouen Cathedral in France.

The object of these observations was to ascertain the difference in barometrical pressure existing between the two ends of a vertical air-column, and to observe how far and in what manner this difference, or the weight of the air-column, changes when the state of humidity of the

* See H. C. Russell, Report on Rainfall of New South Wales.

air varies. At the base and top of the steeple of the cathedral, which is upwards of 500 feet high, was established a station containing quicksilver barometers (and aneroid barometers to check the readings of these) and dry and wet bulb hygrometers. At convenient places between the two

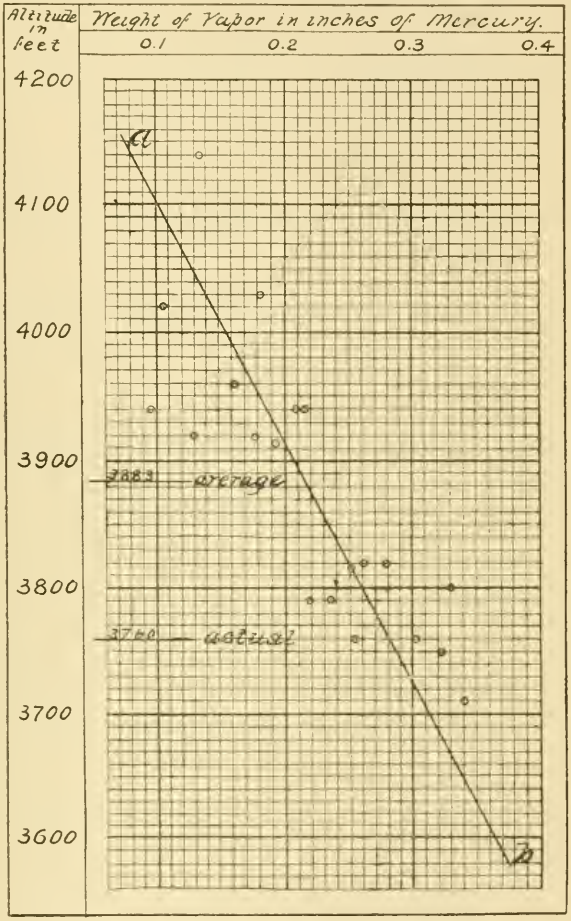

Fig. 1.

stations thermometers were hung out, with the object of attaining a fair average of the temperature of the air-column. The two stations were connected with a telephone; and at convenient hours, at any time during the day or night, for a period of about fourteen days, synchronous readings of all instruments were taken at both stations.

The instruments were the best and newest made by Messrs. Negretti & Zambra, of London, and were all adjusted at the Kew Observatory. It is my pleasant duty to mention here that this well-known firm of instrument-makers wrote me a polite letter, in which they offered the loan of their instruments, free of charge, considering the interest involved in my researches. To assist me in making these observations, I secured the services of Mr. McClellan, of the Greenwich Observatory, who has had many years' experience in handling the most delicate meteorological instruments. Thus everything possible was done to obtain reliable observations, and the result was as stated below. Instead of reproducing here the figures of my own personal observations, I think it will be more to the purpose to point out how the same result can be distinctly traced in the observations made by others, at a time when they were not aware that these results could be deduced from their observations. As such, I have selected Professor S. P. Langley's "Professional Papers, Signal Service No. XV, War Department, U. S." (See Note II.)

The results of my observations showed that an air-column 150 meters high, between the top and base of this tower, became 1.3 per cent. lighter by an increase of atmospheric humidity, indicated by an increase of elastic force of vapor of from .2 to .3 of an inch pressure. The temperature was reduced to 40 degrees Fahr., and the atmospheric pressure to 30 inches.

Regnault has decided that free air is a vacuum to moisture and will hold just as much as a vacuum, but this does not imply that 1 cubic foot of dry air by being intermixed with 1 cubic foot of saturated vapor should form 1 cubic foot of saturated air, air-pressure and temperature being equal. According to Glaisher's "Hygrometrical Tables," 1 cubic foot of dry air, at 40 degrees and at a pressure of 30 inches, weighs 557.8 grains, while 1 cubic foot of saturated air weighs 556 grains, and 1 cubic foot of saturated vapor weighs 2.86 grains. By mixing these equal volumes of dry air and vapor, we obtain a total weight of $557.8 + 2.86 = 560.66$ grains. If this mixture be saturated air, its volume will be $\frac{560.66}{556} = 1.008$ cubic foot. The dry air, by becoming saturated, has therefore suffered an expansion of 0.8 per cent.

According to my observations at Rouen, the loss in weight would, under similar conditions, have been 3.2 per cent., or four times greater than shown by the tables.

How are these seemingly contradictory results to agree? The method of taking the weight of a certain volume of air confined in a vessel, by which the tables have been computed, is eminently adapted to give us the exact specific gravity; and the experiments have been repeated so often by excellent observers that we have no reason to doubt their correctness. If, therefore, we take it for granted that only one-fourth of the loss in weight sustained by the open column of air was due to expansion, the rest, or three-fourths, must have been due to the buoyancy of the aqueous vapors, which would cause friction while tending upwards through the atmosphere and thereby carry a part of the weight of the air-column; and this force could under no circumstances have shown itself under the experiments with air in confined vessels, whose absolute weight is taken in a vacuum. (See Note III.)

We have hereby been able to demonstrate the buoyancy of aqueous vapor in the atmosphere as a force that must influence the readings of the barometer very considerably, and we now understand fully what the main reason is why the readings of the barometer are lower when the atmosphere is moist than when it is dry, as in an anti-cyclone or air-cushion, simply because the greater amount of vapor in the moist air exerts a greater friction by tearing upwards through the atmosphere and therefore carries a greater portion of the weight of the air-column overhead than when the air is in a dryer state.* (See Note IV.)

That this method, however, is not strictly mathematical, may be seen by the following instance: Supposing we admit that a particle of air moving toward or from the pole was deflected eastwards or westwards owing to the earth's rotation, then another force of equal importance would simultaneously come into play. In the same degree as the speed of rotation changes while the particle of air is moving from latitude to latitude, the centrifugal force exerted upon the particle of air would change, and consequently any pole-bound current would be deflected into an upper current, and any current in the opposite direction would necessarily be bound to follow the earth's surface. These phenomena, however, do not agree with observations, but it seems hardly in accordance with the strict rules of mathematics to pick out one component of a force, because it seems to suit our immediate purpose, and

* We see now the reason why the effect of this buoyancy has not appeared before in tables as that of Langley. The column of relative humidity shows nothing. It is first by taking the actual amount of vapor in the air per unit of volume that this force becomes obvious. It is measured by this number and not by the relative humidity.

entirely neglect the other component of this force, because it does not come in very conveniently.

The atmosphere being a perfect mechanical mixture of air ($O + N + CO_2$) and aqueous vapor (H_2O), the buoyancy of the latter must mainly depend upon the difference in specific gravity between the vapor and the air by which it is surrounded; and we will now examine how this changes for different temperatures and at different levels.

The figures in the three first columns of the following table are taken from Glaisher's "Hygrometrical Tables." Column "Dry Air (a)" from Table IV, and column "Vapor (v)" from Table VI. By placing these two columns alongside of each other we are trying a new combination which is of greatest importance to our present subject.

WEIGHTS IN GRAINS OF ONE CUBIC FOOT OF (1) DRY AIR AND (2) SATURATED VAPOR, BOTH AT 30 INCHES PRESSURE.

Temperature, Fahr.	Dry Air. (a)	Vapor. (v)	Ratio. $\frac{a}{v}$
0 Degrees.	606.4	0.55	1 100
10 "	593.4	0.84	707
20 "	581.1	1.30	447
30 "	569.2	1.97	289
40 "	557.8	2.86	195
50 "	546.8	4.10	133
60 "	536.3	5.77	91
70 "	526.2	8.01	66
80 "	516.4	10.98	47
90 "	507.0	14.85	35
100 "	497.9	19.84	20

It will be seen from this table how much lighter vapor is than air, and that the difference in specific gravity is highly increased as the temperature sinks. While air follows Gay Lussac's law by expanding by heat and contracting by cold, the vapor follows a law the reverse of Gay Lussac's by contracting by heat and expanding by cold (compare columns a and v), but at a much greater rate for equal differences in temperature; $\frac{v}{a}$ gives us the specific gravity of vapor, that of the surrounding air being 1; it shows how the buoyancy of vapor $\frac{a}{v}$ is strongly increased as the temperature sinks. At 100 degrees the specific gravity of vapor is one-twentieth of that of the surrounding air, and at 0 degrees it is less than one-thousandth part of it. The weights are measured

under a pressure of 30 inches, the ordinary pressure at the earth's surface. To find the weights at higher levels, where the pressure is less, we have only to multiply the numbers of columns (a) and (r) with the same factor according to Marriott's law. As $\frac{v}{a}$ hereby remains unchanged, it appears that the buoyancy of vapor in the atmosphere depends entirely upon the existing temperatures, and is independent of the pressure, or the level at which the vapors are found. As the temperature constantly sinks as we rise in the atmosphere, the buoyancy of vapor, or the force with which vapors tend upwards as they rise to higher levels, is constantly increasing, and at an astonishingly high rate. While, therefore, the speed with which vapors rise in the atmosphere may be more or less imperceptible at the ordinary temperatures at the earth's surface, it is rapidly increased as the vapors rise, and may attain an almost inconceivable magnitude in the extreme cold which exists at a great distance from the earth's surface.

With the results deduced from the table fresh in our mind, we may now draw a picture from nature while trying to follow the vapors on their upward passage through the atmosphere, and we shall see how far our calculations agree with the natural phenomena. To take a distinct case before us, let us suppose that on a fine day, with high barometer, we are in a dry locality in which is found an isolated swampy place or lake (Fig. 2). While the surface air is dry generally, we find it moister over the swampy place, as the sun and the warm and dry air which passes over it cause a strong evaporation to take place. The warm surface air, though expanded by heat, moves over the ground without rising. It is first caused to ascend by being intermixed with the vapor-particles. According to their buoyancy, the vapor-particles tend upwards through the atmosphere, thereby carrying the air with which they are intermixed upwards also, and the ascent of a current of damp air is established. The vapor is the real cause of this motion, each and all of its particles acting as so many minute balloons. Some eight or ten thousand feet overhead, perhaps at a little distance laterally from the moist ground, according to the direction in which the air moves over the ground, we notice an enormous cumulus-cloud being formed, and we have no doubt whatever that it is caused by the current of damp air ascending from the moist piece of ground. The ascending current, after having passed through the heated surface air, gets suddenly into a

much colder stratum, and condensation takes place by mixture of the rising damp air with the cold air it is passing through. As a rule, the chilling caused by the expansion of the ascending current of air gives it a temperature pretty nearly the same as that of the air through which it is passing, and this constant adjustment of its temperature to the surroundings is a most important feature in the atmospheric machinery, as

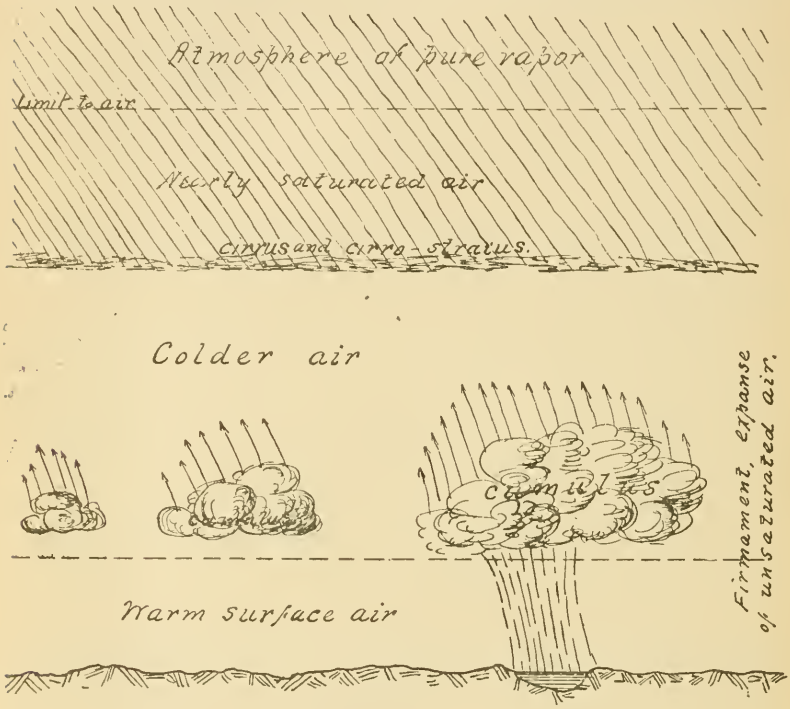

Fig. 2.

without it the vapors would no sooner have commenced to rise before they were stopped in their upward passage by being turned into clouds by mixture. As it is, it is only when the rising air is met by a sudden change in the temperature of its surroundings that condensation takes place by mixture, which we may express by saying that the ascending current has "caught a cold." Instead, therefore, of its being the chilling by expansion which causes condensation into clouds and thereby rain, we see what an excellent preventive against the forming of clouds this very chilling is during the upwards passage. The atmospheric mechanism

is admirably adapted to permit the vapors to rise through the air, and the cumulus-clouds only indicate certain local imperfections in this mechanism, which, however, very soon adjust themselves. We do not, as a rule, expect rain from these enormous cloud-mountains—they are essentially rainless clouds. If the colder stratum of air be moving along, we may notice a row of detached cumulus clouds at some distance from the one nearest to the moist piece of ground, but they grow smaller and smaller the farther away they pass. They are cut off from the supply of damp air, and being surrounded by unsaturated air on all sides, and exposed to the sun's rays, they rapidly evaporate.

The formation of these cumulus-clouds was therefore only a passing event in the ascent of the current of damp air; and as the vapors rose before they were condensed, so they will rise again when they are once more turned into invisible vapor, and the more quickly, the faster the temperature sinks during the ascent. As said before, air and vapor are equally expanded by decrease of pressure during ascent, but the decrease of temperature acts differently upon them, having the effect of contracting the air, while the vapors are very much expanded. For both these reasons the buoyancy of the vapor is increased during the ascent. The vapors must therefore necessarily rise as long as there is any air to pass through, unless they meet with a layer of saturated moisture, or air saturated with moisture.

The clouds produced by the ascent of a current of damp air are cumulus-clouds, and they resemble in their shape very much the mist caused by steam escaping from a chimney. The phenomena are, in fact, precisely similar; and the cumulus-clouds are in their nature as unstable a product as the mist from a chimney, only the first phenomenon is on a much larger scale, and consequently it takes a much longer time for the cumulus-clouds to evaporate. The formation of this class of clouds has, however, had a very disturbing effect upon the conclusions arrived at as to the cause of rain, particularly as they are not absolutely rainless, but occasionally give a shower of rain. A shower, however, is a secondary phenomenon, which does not explain the cause of the great rain, day's rain, or cyclone-rain, capable of yielding 8 to 10 inches of rain *per diem*.

To continue our sketch from nature, at a considerable distance overhead we will generally, on such a fine day, notice some extremely thin and airy clouds—the so-called mare's-tails, cirrus, or cirro-stratus. The

sky is often suddenly changed from a perfectly cloudless one to one completely covered by a thin layer ("pallium," or cloak, as it has been called) of clouds; and records show that not only has it been the case with the small part of the sky we can observe from one place, but that the sky has been suddenly covered by these clouds far and wide for thousands and thousands of square miles.

As to the height at which these clouds are found, I have particularly asked Mr. Glaisher, who is famous for his wonderful balloon-ascents. He told me that he had gone up five and six miles, and passed through other clouds, but he never seemed to get any nearer to the cirrus-clouds. He even went up seven miles; but then he became senseless and unable to observe anything. To estimate their height at thirty or forty miles seems, therefore, hardly to be an exaggeration.

What is the cause of these clouds, and where do they get their supply of vapor to keep them permanent often during the whole long day in the face of the shining sun? As they are strata-like, and entirely different in their shape from cumulus-clouds, we may feel certain that they are not, like the cumulus-clouds, caused by the ascent of damp air. But if their supply of vapor is not to be found below them, it must be on the other side of the strata, or above them. The phenomena of cirrus-clouds is therefore an unquestionable proof of the existence of a uniform and far-stretching layer of saturated air at an exceedingly high level.

Our table has shown us clearly where the invisible vapors rising from the ground must go to, and the cirrus-clouds now show us where they are stored up at a great distance from the earth. Our only difficulty now is to explain how the vapors are brought down from this high level or how they become condensed into clouds and rain. For all we know, a cyclone is a body of surface air brought into rotary motion, and the effect of this rotation is that rain occurs, if anywhere, at or toward the center of the rotating body of surface air, and this takes place whether the cyclone passes over the sea, over moist ground or over dry land. The centrifugal force sweeps the surface air from the center of the

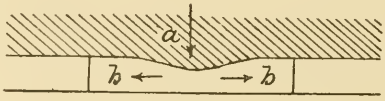

Fig. 3.

a, moist air; *b*, rotating body of surface-air.

cyclone. The partial vacuum which is thereby produced can only be filled up by the descent of the thinner air above the rotating body of surface air. (See Fig. 3.) This is by descending brought under greater pressure; and, as the experiment referred to above shows that condensation or rain can be produced by compression of saturated air, we have hereby arrived at a *possible* explanation of cyclone-rain. This theory agrees with such a general observation as that the clouds are at their lowest level when rain comes from them. (See Note V.)

In contradistinction hereto the rising-current theory implies that the rising current should continually carry the clouds to higher levels during cyclone-rain; indeed the logical consequence is that during a long-continued cyclone-rain the sky should become so obscured by accumulating cumulus-clouds as to change day into night. Instead of that the sky is covered with a uniform layer of stratus-clouds, often of a very thin appearance, and permitting the sun's light to penetrate. Underneath the stratified rain-clouds the process of formation of cumulus often takes place during the rain, and this has perhaps given rise to the complicated name for rain-clouds: cirro-cumulo-stratus-clouds; so we here meet this disturbing element once more.

Likewise in anti-cyclones over the oceans, where evidently a very lively process of rising currents of damp air takes place during the day-time, we should expect a very considerable rainfall to be the rule, but it never rains inside the areas of anti-cyclones.

As rain is generally accompanied by low barometer it may at first sight seem strange that rain should be produced by saturated air being exposed to greater pressure. But if we suppose, as no extraordinary case, that a layer of saturated air is by the action of the cyclone caused to sink from an elevation of 10 000 feet to an elevation of 5 000 feet from the ground, the difference in pressure whereto this air is exposed by far exceeds the difference in barometrical readings at the earth's surface inside and outside the cyclone.

The cyclone causes a hollow to be made in the firmament or surface air; into this hollow the vapors of space are bound to sink and by being intermixed with the thin air form saturated air which gives off rain by becoming compressed, and as the supply of vapor of space is practically unlimited, as will be explained presently, there is so far no limit to the amount of rainfall as long as the ejecting motion of the surface air continues.

In agreement with this *compression theory*, we may explain the prevalence of rain on the windward slope of coast mountains, or mountain-ridges in general. While rain falls at the center of a cyclone, the sky at some distance from where the rain falls is in a condition not far from giving off rain, and so the extra pressure brought to bear upon this saturated air by meeting an obstacle, such as a mountain-ridge, causes rain to set in. The sketch may represent in section a mountainous coast, against which the moisture-laden clouds are driving from out at sea. The current of air, by meeting this obstacle, is caused

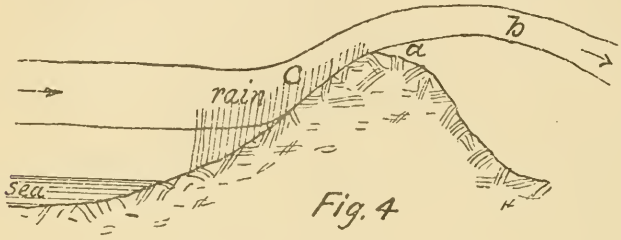

Fig. 4

to rise, following something like the course shown in Fig. 4. At *a* there is always shelter during such a gale, as for instance shown by the sheep and cattle which gather there. In other words, the current rises to its maximum height at *b* above the leeward slope of the mountain. Consequently that is where, according to the theory I am opposing, we should expect the greatest downpour; but there is generally next to none, while the rain nearly all falls on the front side of the mountain. At *c* the current is forced out of its horizontal direction, but a force can only be communicated to an elastic body like air by compressing it. The rain has therefore been caused by compression of saturated air.

Fig. 5 represents a section of the atmosphere of the Southern hemisphere during the month of July, the section being made through the Tropic of Capricorn (see map in my paper, "On the Cause of Trade-Winds"). The intersected parts of the three Southern continents are at that time of the year in a dried-up state, and the air-cushions which consequently develop over their surfaces are thrown westward over the oceans. The height of the vapor atmosphere over the surface of the earth is varying and at its maximum in the anti-cyclones. If we should imagine for a moment that there is no surface evaporation, or that the earth is

perfectly dried up, the air would arrange itself in a uniform layer between the earth and the vapor atmosphere as a continuous air-cushion or shell of uniform thickness, and there could be no possibility of rainfall. The tendency of the vapor atmosphere is towards such a regular shape, but

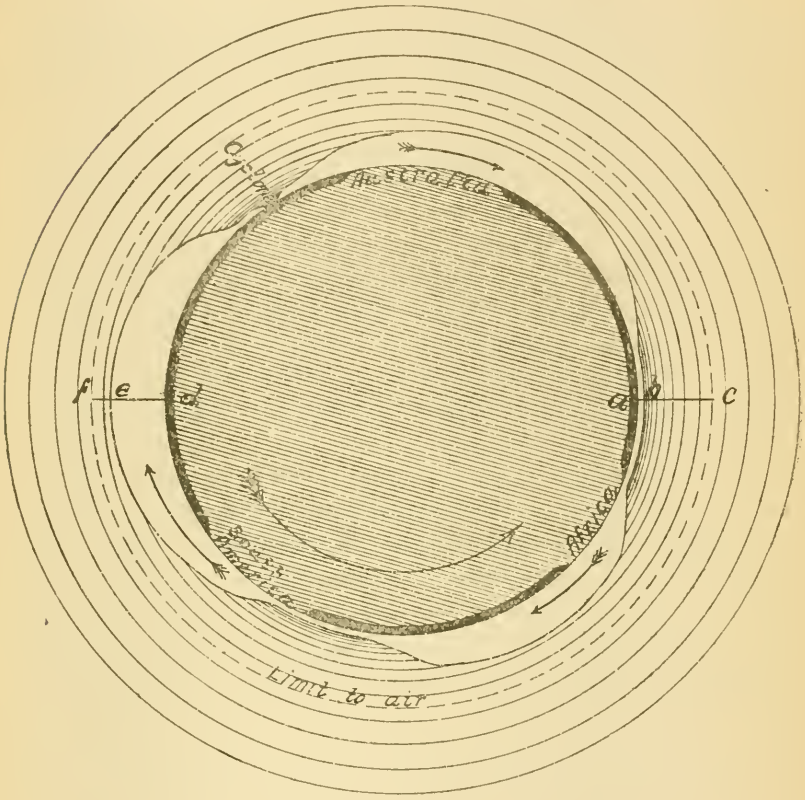

FIG. 5.—IDEAL SECTION OF ATMOSPHERE AT THE TROPIC OF CAPRICORN.
(The circles indicate the outer atmosphere of invisible vapor.)

this tendency is counteracted by the varying degree of evaporation at different parts of the earth's surface. A strong surface-evaporation has the effect of decreasing the height of the vapor atmosphere over the surface of the earth, while little or no surface-evaporation has the oppo-

site effect.* In the space between the anti-cyclones the height of the vapor atmosphere over the ground is comparatively small, and it reaches a minimum when (say, for instance, in the V-depression between two anti-cylones) the surface air is, by the currents of opposite directions along the borders of the anti-cyclones, brought into rotary motion, which drives the surface air away from a center. The vapor atmosphere is thereby caused to approach the earth's surface, and by thus descending is brought under greater pressure, so as to give off rain at the center of the cyclone, as explained above.

Having, by a simple way of reasoning, arrived at the conclusion that an atmosphere of pure aqueous vapor must exist outside the atmosphere proper, we should not feel justified in stopping without carrying our idea out in at least some of its consequences, although the following remarks do not concern our immediate subject, the cause of rain. Supposing there was an outer limit to this aqueous atmosphere, the difficulty which would present itself is, that we should find aqueous vapor alongside of the vacuum of space. It is well known that when moisture is brought into an artificially produced vacuum, the latter gets instantaneously filled with aqueous vapor. How is this experiment to agree with the popular notion that vapor, as well as the other constituents of the atmosphere, is kept within limits round the earth by means of gravitation? If the vapors of the supposed outer border of the atmosphere were prevented from entering space owing to gravitation, how much more would the vapors at the bottom of an artificial vacuum be prevented from filling this space, as the force of gravity is much the greater at the earth's surface than at a supposed outer border of the atmosphere? Surely, we must conclude that it is impossible to imagine how moisture could remain in the air or on the earth's surface, unless space were filled with aqueous vapor. The earth's surface being practically that of a huge drop of water, and this drop moving round the sun in a supposed vacuum, how could this moisture be prevented from escaping into space unless space was filled with aqueous vapor? The only thing to prevent such an emergency is the thin veil of an atmosphere; but this, being itself all permeable and permeated with aqueous vapor, seems indeed a very poor protection.

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* An air column, as *a b c*, is composed of a short column of dry surface air, *a b*, superimposed by much moist air, *b c*. In the column *d e f*, the proportion between moist and dry air is reversed; *a b c* shows low pressure, and *d e f* high pressure, according to the principle of buoyancy of vapor.

Laplace's nebular theory of the evolution of the solar system points toward the same fact; for, if aqueous vapor has once been uniformly dispersed throughout the solar space, it follows of necessity that this space could not afterwards have become perfectly exhausted of aqueous vapor; gravitation toward the sun and the planets could not establish such a vacuum.

If it should be used as an argument against my theory, that we might with just as much right expect to find the other constituents of the atmosphere dispersed through space in a rarefied state, then I would say, as has been pointed out above, that these follow the reverse law of aqueous vapor by being contracted by cold, and that makes all the difference.

But space being all filled by aqueous vapor, how is it, we may be asked, that this fact has entirely escaped observation? By being situated with their instruments in the midst of it, the atmosphere and space being all occupied by aqueous vapor, our observers are taken rather at a disadvantage. It sometimes happens that people cannot see the forest for trees, namely, when they are standing in the denser part of the wood.

The general conclusion I arrive at is, therefore, that the interplanetary space is filled with vapor in an extremely rarefied state. The sun and each of the planets is surrounded by a vapor atmosphere of a denser state, the quantity of vapor surrounding each of these bodies depending upon its size and its surface-temperature. The sun will for both reasons have by far the lion's share of such a vapor envelope. This theory seems to agree perfectly well with the following observed facts:

1. The retardation suffered by the comet Encke indicates that this comet, when nearest to the sun (that is, at a distance from the sun about that of Mercury), passes through a medium of a certain resistance.

2. The present condition of the surfaces of the four inner planets varies according to their distances from the sun, or what is likely to be in proportion thereto, their surface-temperatures. On Mars we find more land than water surface, and a clear sky. The conditions on the earth in this respect need not be repeated here. As to Venus and Mercury, they possess an atmosphere of great density; and, as they are constantly covered by clouds, we have no means of ascertaining the proportion between land and sea surface, but their clouded state seems to indicate that they must be entirely or almost entirely covered by

water. These varying conditions seem to indicate that the planets are gradually approaching a state of being dried up, or that their waters and vapor envelope are gradually leaving them; and the conditions on the moon indicate that this state will have been reached when they have become extinct planets.

3. The moon being an integral portion of the earth, there can be no doubt but it must once have possessed surface waters and a vapor envelope in proportion to its size. It is now an extinct planet, and its surface is void of waters. What has become of this water, unless it has passed into space?

We have hereby gained a fresh point of view, from which it may be worth our while to reconsider the former, present and future conditions on the earth. Geologists have come to the conclusion that at the time of the coal period there must have been much less land surface than now, and that the atmosphere must have been much warmer and moister than it is now. The land may, of course, gradually have emerged from out the seas since then, the quantity of water on the earth remaining constant; but it seems exceedingly more natural to suppose that the earth contained much more water during the coal period than it now contains. When, therefore, we nowadays find ancient sea-beds in the highest of mountain-ridges, we need not feel so sure that these have risen to their present elevated position from under the present level of the sea, as probably the sea level was formerly quite different from what it is now.

Mathematicians have at various times attempted to determine the outer limit of the atmosphere by calculating at what distance from the earth there would be equilibrium between the centripetal and the centrifugal forces acting upon the smallest particle of air, thereby arriving at results varying from fifty to two hundred miles, the difficult point being how to determine the actual mass and density of the particles of air. Other philosophers have seen the absurdity of imagining the situation of an air-particle in a state of uncertainty as to whether to remain with the earth or go off at a tangent, and therefore have concluded that the atmosphere is practically unlimited. Another series of considerations, based upon optical observations, has led to the conclusion that outside the atmosphere of air, which may be estimated at forty or fifty miles, must exist an atmosphere exceedingly thinner than air. These various theories are brought into perfect

agreement, and the absurdities are avoided, by assuming my theory of an outer atmosphere of vapor which is unlimited.

It is truly said that there is nothing new under the sun, and we might therefore expect to find that my definition of the atmosphere is merely a repetition of what has been said at former times. In his excellently written book, Mr. Scott points out that the old Biblical Scriptures, particularly the Book of Job, contain many a sound reasoning on the atmosphere which holds good to this very day. Not having found the information I wanted in modern works on this subject, I took the hint and looked up these ancient sayings, until I came to a passage by Moses which made me pause. Perhaps an Ingersoll may some day point out some serious mistakes in my argument or in the figures I have produced to support it, but at present it seems rather as if Moses managed to give us a pretty clear definition of the atmosphere when he wrote: "And God said, Let there be a firmament [Hebrew, "expansion"] in the midst of the waters; and let it divide the waters from the waters. And God made the firmament, and divided the waters which were under the firmament from the waters which were above the firmament: and it was so."

The coincidence between this observation of Moses and the result I have arrived at may, perhaps, in our advanced age, be considered merely as a curiosity; but, considered as a purely objective and perfectly unbiased view of the matter, it seems to me to afford some further interest. Moses could not argue much on atmospheric subjects, as he had no natural sciences to guide him, but neither could they lead him astray. His knowledge of air was very limited. He did not know that it exerts a pressure of 14 pounds per square inch, and that this pressure grew much less when he went on to Mount Sinai to write the Commandments; and neither could he have any knowledge of the existence of invisible vapor. But when he walked about in the desert for forty years under a generally serene blue sky, and on rare occasions saw a cyclone set in, then he would observe this phenomenon exactly as Mr. Scott describes it nowadays: "He saw the thin cirro-clouds overhead gradually change in stratus, and these gradually growing further condensed and sink to a lower level, until rain ultimately set in." He saw the clouds and rain being formed on the spot, and could have no suspicion of their being caused by vapors rising out of the dry sand of the desert; and so he wrote faithfully according to what he saw. Although, there-

fore, no doubt, there are more things between heaven and earth than was dreamt of in the philosophy of Moses, when he tells us that there is a firmament between heaven and earth, dividing the waters from the waters, the time may perhaps not be far distant when we shall all agree with him on that particular point.

NOTE I.—The trouble with this experiment has always been that by compressing the saturated air the condensation of moisture is partly due to contact between the cold inner surface of the glass cylinder, which continues at the air temperature, and the compressed air, which by compression obtains a higher temperature. Many people are apt to conclude from this that the condensation is entirely due to this circumstance, particularly so, perhaps, as saturated aqueous vapor or steam is not liquefied by compression. But there is a great difference between saturated air and saturated aqueous vapor, and it really seems a pity if it some day should appear that meteorologists have come to grief by making an unsafe or erroneous deduction from Regnault's experiments on saturated vapor instead of examining saturated air for themselves. As the proper study of mankind is man, so it would appear that the proper study of the atmosphere is air and not vapor or ether or any other entirely different substance.

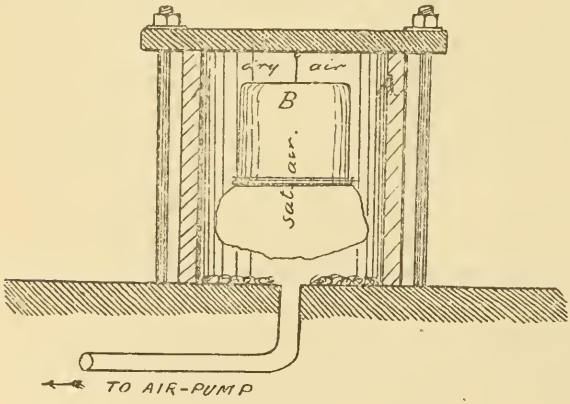

FIG. 6.

Although now it undoubtedly follows from the experiment that condensation takes place by compression of saturated air, because it shows

that expansion has decidedly the opposite effect, I may recommend the following experiment to those who doubt until they see for themselves.

A strong glass cylinder, *A*, with air-tight fitting top and bottom, is in connection with an air-pump capable of conveying a pressure of two or three atmospheres to the interior of the glass cylinder. In *A* is suspended a vessel, *B*, of as thin glass as it is possible to make it, and to the edge of *B* is fastened a cover of thin India-rubber or other suitable material, the volume of air contained in this cover being about equal to that contained in *B*. The air within *B* is kept in a state of saturation by applying a little moisture to the inside of the India-rubber, and to secure that this air is actually saturated at the commencement of the experiment, it may be as well just before placing *B* in its position to raise its temperature a few degrees, as then, while it cools down when hung up in *A*, a small quantity of dew will be deposited on the inside of the vessel, *B*, just enough to show that this air is saturated.

As now the temperatures in *A* and *B* are exactly the same, they will practically remain equal to each other while air is pumped into *A*, as the increase of pressure in both vessels is exactly the same, and as *B* can be made of as thin glass as possible, it having absolutely no difference in pressure to withstand, it is secured that its temperature is gently raised by the temperature of the air surrounding it on both sides being gently raised. Any mist formed or dew deposited within *B* must therefore be owing to the compression of the saturated air it contains.

To prevent any dew being deposited on the inside of *A* so as to obscure the view of *B*, the air in the vessel, *A*, is kept dry by some moisture-absorbing material, such as chlorate of lime, being placed at the bottom of *A*.

I have enlarged on this subject because the peculiar notion has found favor in certain quarters that a body of air should become drier by descending in the atmosphere, and consequently be the cause of the dryness of the air in high pressures. But no engineer who takes the map in my paper "On the Cause of Trade-Winds" in hand, will for a moment doubt that the general circulation of the surface air which takes place over such an extensive area as that covered by the north Atlantic anti-cyclone is that of a horizontal current of air, as horizontal as the surface of the land and sea over which it passes will allow it to be.

NOTE II.—On page 191 of Prof. Langley's paper is a table showing the results obtained by measuring the altitude between sea-level and Lone Pine, Mount Whitney. Of these forty measurements, I have in the following table given the ten highest (upper half of table) and the ten lowest (lower half), arranged according to the height.

Alongside of each of these figures will be found a number representing the weight of vapor per unit of volume contained in the atmosphere at the time of observation. These latter numbers are obtained by multi-

TABLE OF BAROMETRIC MEASUREMENTS OF ALTITUDE BETWEEN SEA-LEVEL AND LONE PINE, MOUNT WHITNEY.

Time of Observation, 1881.	Results in Feet.	Weight of Vapor in Inches of Mercury.	At Lone Pine.		
			Relative Humidity.	Temperature. Fahr.*	Wind.
August 26th, noon...	4 140	0.1354	10.3	87.8 degrees.	Brisk.
August 26th, 9 P.M....	4 030	0.1823	28.8	65.7 "	Fresh.
September 3d, noon..	4 020	0.1060	8.7	85.4 "	Gale.
August 29th, noon...	3 960	0.1627	15.6	80.6 "	Fresh.
August 25th, noon....	3 940	0.2081	18.1	83.6 "	Fresh.
September 3d, 9 P.M..	3 940	0.0960	13.5	69.1 "	Brisk.
August 24th, noon....	3 940	0.2158	16.0	88.6 "	Fresh.
August 17th, noon....	3 920	0.1782	14.9	84.6 "	Brisk.
September 2d, noon..	3 920	0.1302	9.9	87.8 "	Brisk.
August 24th, 9 P.M....	3 915	0.1931	29.0	67.2 "	Brisk.
August 20th, 9 P.M....	3 820	0.2787	35.5	72.0 "	Gentle.
August 19th, noon....	3 820	0.2615	17.8	91.3 "	Gentle.
August 23d, 9 P.M....	3 815	0.2508	38.3	66.7 "	Fresh.
August 18th, 9 P.M....	3 800	0.3314	78.0	54.7 "	Gentle.
August 31st, 9 P.M....	3 790	0.2190	45.7	57.6 "	Calm.
August 30th, 9 P.M...	3 790	0.2352	53.1	55.6 "	Calm.
August 27th, 9 P.M...	3 760	0.2543	68.0	51.0 "	Calm.
August 21st, 9 P.M....	3 760	0.3034	38.4	72.2 "	Brisk.
August 22d, 9 P.M.....	3 750	0.3232	70.1	56.7 "	Fresh.
August 19th, 9 P.M....	3 710	0.3405	71.5	57.6 "	Gentle.
September 7th, 9 P.M.	3 625	no record.			Calm.
September 6th, 9 P.M.	3 620				Calm.

* The ten highest give an average of 80 degrees Fahr.; the ten lowest, 63 degrees Fahr.

plying the relative humidity at Lone Pine (see table in Langley, page 177) by the elastic force of vapor (Glaisher's tables).

The difference between the maximum and minimum result is 520 feet, or 14 per cent. of the trigonometrically surveyed height, 3 760 feet. This latter number was obtained by the engineers who built the railroad passing Lone Pine. Langley's party wrote to the engineers for this information, and awaited their reply with considerable anxiety.

From the table it will be noticed that the amount of vapor in the atmosphere was considerably less when the ten highest results were obtained than when the ten lowest were obtained; and, as perhaps may be better illustrated by the accompanying diagram (Fig. 1), there appears to be an unmistakable relation between the measured heights and the humidity of the atmosphere. The heights are here placed at distances from the vertical line to the left proportional to the amount of vapor in the air, and the line, *a b*, shows the general tendency of the figures. That these do not follow the line, *a b*, more closely, may be accounted for by the humidity of the air having been measured only at one end of the air-column, namely, at Lone Pine. Besides, the method of determining the relative humidity by dry and wet bulb thermometers is highly unsatisfactory, the results being very much affected by the rate at which the wind carries the air across the wet bulb. To secure at all reliable

results the thermometers ought to be perfectly enclosed in a chamber, through which the outer air is caused to pass at a perfectly uniform rate, for instance, by means of a pump, there being wind or no wind.

We may, however, trace another coincidence in our table. It will be noticed that the ten highest results are generally from observations taken at noon, when the thermometers showed a relatively high temperature, while the ten lowest results are all (except one) from observations taken at 9 P.M., when the temperature was considerably less. This is only what we might have expected, and shows that the buoyancy or tending upwards of the vapors in the atmosphere is considerably greater at a lower temperature than at a higher, as will be explained in the text.

Professor Langley's observations give results corresponding to those I obtained at Rouen, and the relations here pointed out may be traced in numerous works from ancient and modern times, though perhaps not in all. However, it should be remembered that it is not so much my present purpose to show how the measurement of altitudes by means of barometers may be carried out with greater accuracy than hitherto (this method being highly unsatisfactory for obvious reasons) as to show that the hidden agencies which are at work in the atmosphere, and without assuming which the whole atmospheric problem remains unsolved, may be distinctly traced in the observations carried out by others. Surely we must expect to find the secrets well concealed, or they would have been demonstrated ages ago; but here, as elsewhere, it is the instances when "the sky is unobscured by clouds," to use a figure of speech, which we must select to make our observations, and Langley's table is such an instance. The objection might be raised against my utilizing Langley's figures in the way I have done, that his principal object was not to determine the height of Lone Pine above sea level, and that the horizontal distance between the two places of observation is so great, 160 miles, that no close determination of this height was to be expected. This I quite admit, but this does not account for the close connection, which I have pointed out to exist between his results as to height and the amount of moisture in the air.

In Williamson's work, "On the Use of the Barometer," and in all previous works on the subject, the effect of aqueous vapor in the air is estimated to impart additional weight to the air-column. Of late years this plan appears to have been abandoned as not quite reliable, and as Professor Langley, in his measurements of mountain-heights, here referred to, leaves the factor due to vapor in the air quite out of consideration in calculating the weight of the air-column, his table becomes particularly suitable to demonstrate that the actual effect of the vapor in the air is to make the air-column lighter than the corresponding column of dry air.

NOTE III.—As the principle here involved appears new to me, it may require some further explanation. If we fill a U-tube with a fluid and

keep a lighter body submerged in one of its branches, the fluid will stand at the level in both branches as long as the submerged body is at rest; but the situation becomes different when the body is in motion or allowed to move upwards by being set free. What then takes place may best be illustrated by the following experiment: We fill the U-tube with an effervescent fluid, such as soda water or champagne, and notice that it stands at the same level in both branches after having settled down. If we then gently stir the fluid in one branch so as to cause effervescence to take place there, we notice that while air bubbles are moving upwards through the fluid of this branch, the fluid in the other undisturbed branch sinks to a lower level. We learn, herefrom, that the pressure at the bottom of the U-tube has become less, and this diminution in pressure can only be due to the friction between the air bubbles and the fluid through which they pass upwards.

It was by considering the diurnal oscillation of the barometer, that it first occurred to me that the action of vapor particles rising through the atmosphere must have a similar effect to that of air particles rising in the U-tube; and we may now proceed to show that this extraordinary phenomenon is easily explained by applying that principle.

In a letter to the Editor of *Engineering*, January 11th, 1889, the writer said :

"The Diurnal Variation of Pressure," says the Hon. Mr. Abercromby, on page 310 of his recently published work, "Weather," "has two minima at 4 A.M. and 4 P.M., and two maxima at 10 A.M. and 10 P.M. respectively. The single diurnal variations are undoubtedly due to the direct influence of the sun's heat; but the question how an influence such as that which runs its course only once in the twenty-four hours can induce a variation which has a semi-diurnal period, has, up to the present time, baffled the skill of meteorologists. It is, however, perfectly certain that no one is the cause of the others; all are equally the products of the same influence, and no comprehensive theory of diurnal variations will ever be complete which does not explain and correlate all together."

It is the purpose of the following lines to demonstrate that the combined effect of daily uptake of moisture and expansion by the sun's heat must necessarily produce a semi-diurnal oscillation and no other.

For the sake of simplicity we may assume that the surface evaporation only takes place from the time the sun rises till it sets, and to be quite sure of this we may take before us the case of dew being deposited during the night, as it is pretty clear that evaporation and condensation of moisture cannot take place simultaneously from the same surface.

The surface air becomes expanded by the sun's heat, and by the vapor it absorbs. If a body is lighter than a fluid in which it is submerged, it will tend upwards with a force or pressure equal to the difference between the weight of the fluid it has replaced and its own weight, and if it is allowed to rise through the fluid it will exert a pressure against the

incumbent fluid equal to its force, or what comes to the same, it will, while rising, carry a part of the weight of the fluid above it. If we apply this rule to the present instance of expanded surface air submerged in an atmosphere, it will be seen that this air, while rising through the atmosphere, will carry a part of the weight of the atmosphere, and how much it carries is shown by the falling of the barometer.

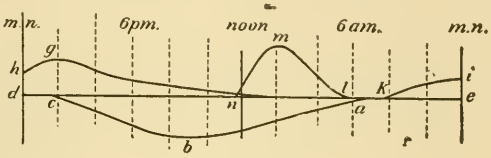

FIG. 7.

This falling is represented in Fig. 7, by the curve *a*, *b*, *c*, the line *d*, *e*, being mean pressure. This curve commences at sunrise, and ends some hours after sunset, when the latest expanded air ceases to rise after having found its place of equilibrium, namely, when the specific gravity of the surrounding air is equal to its own.

The vapors imparted to the atmosphere have, however, a certain weight, and this weight will assert itself as increase of pressure when the vapor particles cease to rise. The curve *f*, *g*, *h*, *i*, represents this increase of pressure on the barometer. It commences several hours after sunrise, when the first of the rising vapor particles have arrived at their place of rest, and reaches its maxima where the curve *a*, *b*, *c*, ends. From this point the curve is gradually descending until sunrise as the uptake of vapor in the atmosphere is constantly given off by rainfall, so during the night, when no fresh supply is received, the pressure caused by vapor must be constantly sinking. At sunrise the atmosphere is in a state of equilibrium as far as the disturbing influences of the sun is concerned; then when the expansion by heat and by vapor causes an extra volume to be introduced along the earth's surface, the surface air must be pushed upwards to give room for this fresh body of air. The new body of air forms a pressure center, and the pressure it exerts against the incumbent air is exactly equal to the increase of pressure at the earth's surface. The first effect caused by the expansion is, therefore, increase of pressure at the earth's surface, and this increase of pressure reaches its maximum when the development of air body per hour is at its greatest. Afterwards, when circulation has properly set in, the volume of air rising equalizes the volume developed during a given period of time, and this increase of pressure is no longer felt. The curve *l*, *m*, *n*, represents the effect on the barometer of this pressure.

If we now combine these three curves we get a resultant curve as shown in Fig. 8; this curve has two maxima and two minima, and they

occur at the same hours as those of the diurnal oscillation of the barometer. Whether other factors, such as the rotary motion of the earth, may have a modifying effect on the oscillation, others may be able to prove, but I think it has now been demonstrated that the effect of the sun's heat would by itself be able to produce an oscillation very similar to the one we find, and that consequently these other factors cannot have any great influence unless they work together with the sun.

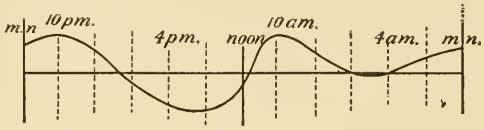

Fig. 8.

When the earth is covered by a dense layer of clouds the upper surface of this layer practically becomes the earth's surface, and the expansion caused by the sun then goes on there. The general run of the diurnal oscillation of a barometer placed underneath the clouds will therefore remain unaltered. On the other hand, it will be obvious that the oscillations are likely to be different, at least as to time, if not also in shape, at the top and the base of a high mountain, and unless this difference is taken properly into account the measurement of a mountain by means of barometers cannot be expected to be perfectly reliable.

We have here explained the diurnal range of pressure as being mainly the effect of the daily uptake of vapor, and it may therefore be worth noticing that it changes in intensity with the degree of surface evaporation. It is greatest in the tropics, and dwindles into a mere nothing toward the poles. It is greater at the coast of large continents than far inland where the land is more or less dried up, but increases when we, from the central part of North America, approach the large Canadian lakes.

Note IV.—This explains why meteorologists of the mathematical school have not been able to bring the atmospheric movements to agree with their calculations. They start from the supposition that centers of high pressure are due to a greater quantity of air being accumulated over these areas than elsewhere, and that consequently the air to re-establish equilibrium will tend to move outwards in all directions from the center, but we see now that the principal cause of the pressure being high is that the air-column overhead contains less moisture than outside the high pressure, and that this moisture while moving upwards through the air acts as a negative weight. Under these circumstances it may appear less surprising that the anti-cyclones dealt with in my paper on "Trade-Winds" can remain permanent nearly all the year round, instead of spreading far and wide in a short measure of time.

NOTE V.—The nomenclature of clouds being a question which of late years has provoked considerable dispute, it seems to me that, according to my explanations of the general atmospheric arrangements, clouds might more properly be grouped according to their course or origin, rather than entirely according to their appearance, which is so varying and deceptive. We find thus two distinct groups of clouds; namely, what we may call "evaporation clouds" and "condensation clouds."

Evaporation clouds are cumulus-clouds in shape. They are formed by mixture at the summit of a current of damp air arising from the ground. They are unstable and merely indicate a stage in the upward passage of vapors. They are essentially rainless clouds, and found in the expansion or cushion of unsaturated air. They have their supply of vapor from below.

Condensation clouds are stratified clouds, cirrus and stratus. They are formed by condensation by compression at the lower limit of the outer atmosphere of vapor. They are essentially rain-clouds, or those from which the great rain ultimately comes. They have their supply of vapor above them.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

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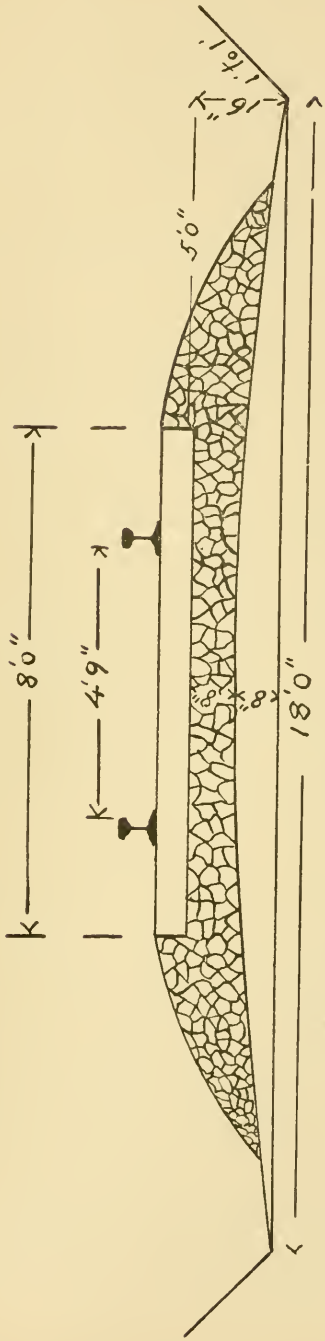
CONSTRUCTION AND MAINTENANCE OF TRACK.

By JULIEN A. HALL, M. Am. Soc. C. E.

WITH DISCUSSION.

There has been so much written, and by so many authorities, on the subject of track, that it is with much hesitation that I endeavor to add to this store of information, but I have collected a few facts from my own and others' experience, and hope that the following pages which contain them will present a few points which may be of interest to some, if not to all, of our members.

The roadbed should first be considered, and the cuts (Fig. 1) for a single-track standard-gauge road should, in ordinary earth, be taken out 18 feet wide at the base, sloping from either side one foot horizontal for each foot vertical. This slope is, of course, more expensive than a less one, but it is more economical in the long run to use the one to one slope, as the cost is usually less to take out the material while the grading is in progress than to do it afterwards, with work trains and section gangs. In rock cuts the width of the roadbed may be reduced to 16 feet, and the slopes to a half or quarter to one, but care should be



RICHMOND AND DANVILLE RAILROAD STANDARD ROADBED SECTION.

Scale, 1" = 3'.

1890.

FIG. 1.

taken that there is no projection from the side of the cut further than to a point not less than 3 feet from all passing trains.

Where borrow pits are made, they should never be taken out below grade of cut in such a manner as to allow for the formation of pools of stagnant water, and in order that this end may be more readily attained, the engineer in charge should always stake out the borrow pit, take elevations over it, and direct the contractor fully and clearly in regard to it, and if this is done it is not probable that the trouble indicated will occur.

Good drainage is absolutely necessary for good track, and great care is always required to obtain this. The center of the cut should always be higher, by not less than 8 inches, than the bottoms of the ditches on either side, and the lateral slopes of the roadbed should extend regularly to the sides of the cut, their intersection with the lines of same being the lowest point in any section of the roadbed, thereby relieving the roadbed to a great extent from any cutting that may occur from flow of water, and concentrating the larger part of this action on the lower edge of the cut. If the cut is through sloping ground, one side of it will be exposed to watershed from the ground above it, and in order to prevent this water from tumbling over into the cut a ditch should be dug about 3 feet from the upper edge of the highest side of the cut, which ditch should be of sufficient capacity to carry off to the nearest water-courses all of the water likely to be shed into it. In digging the ditch the material should be thrown on the side next to the cut, thereby making a dam, which in the case of unusually high water would be useful.

Wet Cuts.—If a cut is dry in dry weather, keeping the ditches clear of the earth which sloughs off of the sides will usually be all-sufficient to insure free passage for water falling on roadbed and on sides of cut, but if the cut is always wet and springy, other methods of getting rid of the water will have to be used. Deep ditches for this will generally be found ineffectual, as this, in conjunction with the ditches at the top of the cut, will sometimes cause heavy slides from the sides of the cut, which often extend entirely across the track and cause accident unless discovered in time to prevent. A very satisfactory way to relieve wet cuts is to lay one or more lines, as may be necessary, of circular terra cotta pipe, about 3 feet below the bottoms of the ditches, or deeper if necessary, to get below the line of frost, and this pipe should be of

sufficient capacity to carry off the water to be gotten rid of. The longitudinal sections of the pipe should be as short as practicable, thereby increasing the number of joints, which should be left open, and through which the water will readily percolate, after which the cut will generally be found well drained and a good track can be maintained therein. But should this fail, special plans best suited for the troublesome places will have to be resorted to. Sheet piling in both ditches, driven through the mud and into the hard underlying clay, will be found effective. Farmers' drains are also very satisfactory; these are composed of two, three or four lines of logs, laid parallel under each ditch, each end of each line terminating in the berme ditch of the nearest fills. If neither of the methods noted remedies the trouble, as a last resort, though an expensive one, build a double retaining wall in the form of a culvert under each ditch, paving the foundation well, which should, of course, be the best obtainable; cover the top, leaving the space in clear between walls about 2 feet, and clear height inside about 3 feet. The material may for a while continue to slough and slide, but the retaining wall, which is only a culvert, will be found to drain the cut properly, and the trackmen will be enabled to maintain a good track therein.

Fills should be made not less than 14 feet wide for a roadbed for a single-track standard-gauge line, and staked out for one and one-half to one slope, and all waste material taken from adjacent cuts should be dumped equally on either side of the fills, thereby increasing their width regularly, and thus strengthening them in a symmetrical and economical manner and avoiding unsightly waste banks. A berme bank from 3 to 5 feet wide should be outlined outside of the slope stakes, and the contractor should not be allowed to borrow material within this limit. Outside of and at the edge of this limit a ditch of sufficient section should be cut, to prevent water from injuring the bank and to carry it to the culvert at foot of bank. The culverts under the banks, unless they are very large, should always be located a little above the greatest depression in the hollow, which will generally prevent the opening from being choked by trash; care, however, should be taken not to locate the opening too high above the lowest point, as such an error would often cause accumulation of water, which might become objectionable to natives living near the point, in which case the company would be very likely to be troubled with suits. Care should also be taken to have the culvert of sufficient size to pass accumulation

from excessive rainfalls or melting of snow, and this matter should be very carefully considered before the size of the opening is decided on; and in excess of the amount decided on, a good factor of safety should be allowed, which will possibly at some time save the bank. Should the opening be too small, a serious washout may occur; and if the fill should wash out at night, unless the track is carefully policed, a serious accident might occur. Should the fill withstand the pressure of an overflow and act as a dam, backing up the water for some distance, this, as well as a washout, will be another source of annoyance from adjacent landowners, and the company generally gets out of this trouble only by paying for so-called damages, from six to ten times the amount of the value thereof. Banks made in layers of about 1 foot or slightly more, will be found to be more solid and to require less allowance for settling than the fills which are made by dumping the material off the end of the bank. Should this latter mode of making the fill be necessary, cart and car work will be found better than wheelbarrow work. The surface on steep hillsides should be well benched out before the fill is begun, after which the earth will be found to catch and hold much better than to allow it to roll to a resting-place.

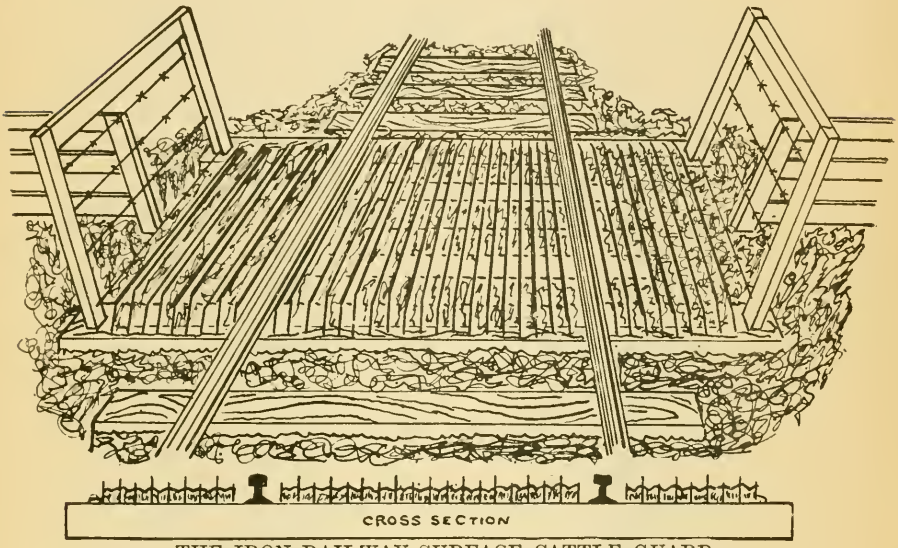
Openings.—These should be as seldom as possible, and no trestles used unless absolutely necessary; but where they are used, they, as well as bridges, should be of iron, and such structures should always be calculated for the heaviest rolling loads to be put over them, and a factor of safety of not less than 5 should always be allowed.

Cattle Guards.—When this character of opening is necessary, the standard should be without a pit, which is so dangerous in case of derailment. The National Surface Guard, Fig. 2, fills this requirement, and for its use no change need be made in the track, save the putting in of two longer ties for connection with the cross fences. This guard is made in three and four sections, and can be put in the track very quickly. The company manufacturing them has recently designed an iron fence which will require less repairs than the wooden one previously used. Where guards with pits are used, Fig. 3, indicates one which is now the standard wooden guard of the Richmond and Danville Railroad, and which has given good satisfaction, being very effective in stopping stock of all sorts. The number of feet, board measure, in this guard is 1 040, and it can be put in place very cheaply.

Cross-ties.—These being such a large and important part of the

track, standard dimensions and timber should be decided on and the inspectors made to adhere to them. There is still a considerable amount of good timber in the United States, especially in the South, which will be used for cross-ties, but statistics show that our forests are rapidly disappearing, from the constant drafts on them. Methods of chemically

Fig 2.



THE IRON RAILWAY SURFACE CATTLE GUARD.

preserving wood will be resorted to or metal must be largely used for ties; but as the first cost for wooden ties, even when creosoted or otherwise chemically preserved, is so much less than for metal ties, the growth of suitable timber should be largely carried on and the reckless waste of our present valuable forests brought to a sudden stop. Cross-ties are becoming scarcer every year, and are also beginning to be more expensive, while railroad mileage is on the increase. However, wooden ties will, for some years, still be used in this country, especially in the southern part of it, and I will only mention a few points in connection with this kind of cross-ties. White oak, post oak and chestnut oak are the best timbers for cross-ties, though pine is largely used in the far South, and when a good long-leaf pine tie can be had it will last for

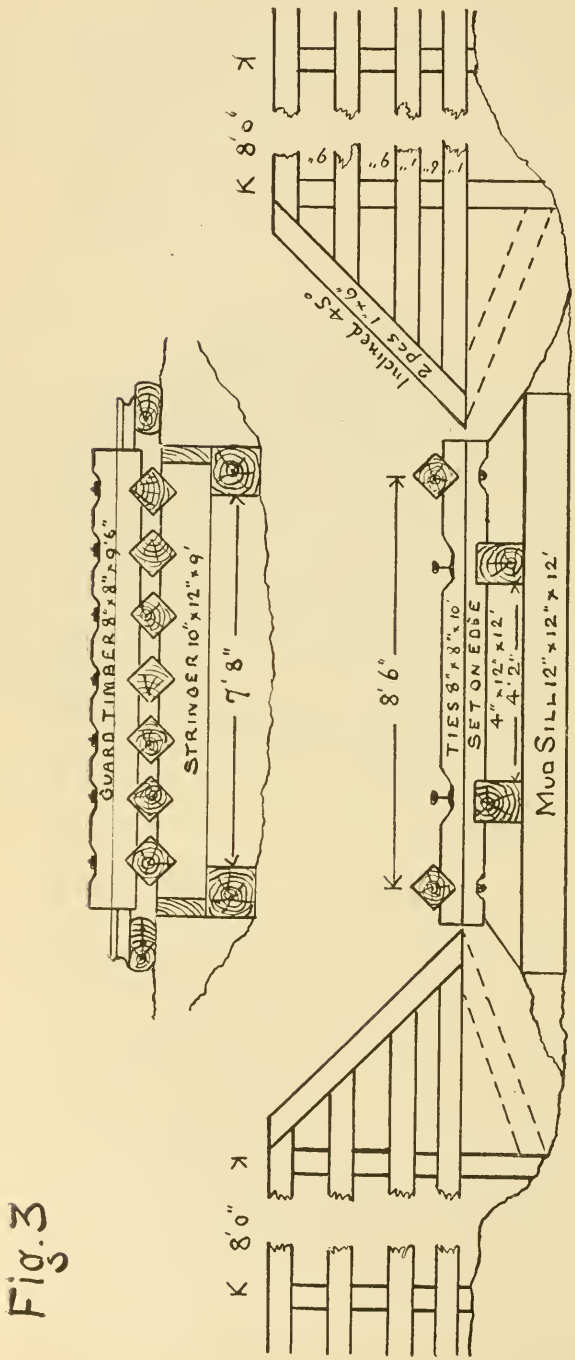


Fig. 3

RICHMOND AND DANVILLE RAILROAD STANDARD CATTLE GUARD.

Scale, $\frac{3}{4}'' = 1'$.

seven or eight years, and sometimes longer, but the average life of the pine tie is not over five or six years, and the habit of bleeding the pine (extracting the rosin), is each year making pine ties more inferior. The original growth, heart pine tie usually rots regularly from the outside, in fact almost wears away, but the bled sap tie will rot anywhere, one end sometimes being good and the other worthless, and will often have the appearance of being sound and at the same time be so rotten as not to be able to hold a spike.

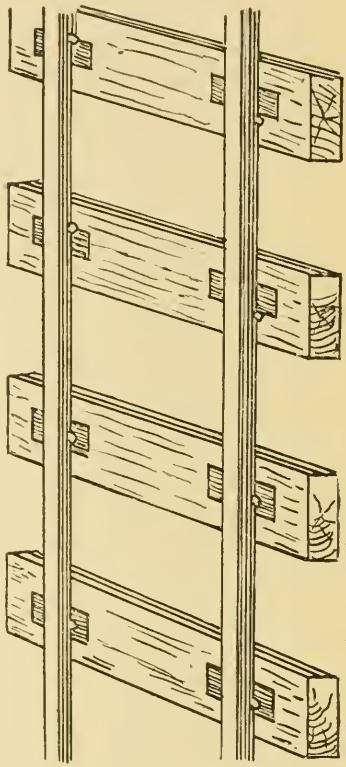
But the oak ties mentioned are very satisfactory, their average life being about six or seven years, and they will, better than any others, resist decay and being cut into by the rail; the latter, occasioned by heavy loads, forms a receptacle for water, which causes rot to begin and soon destroys that part of the tie, making it too weak to hold the spikes and support the rail, and it is rendered thereby unfit for use, though sound everywhere else, except under either rail. Neither chestnut nor any other soft wood is desirable for ties, unless used with a tie-plate, for soft wood ties, owing to the openness of the fiber, soon become unable to hold the spikes, which work loose and sometimes half out, thereby rendering the track unsafe. Ties 8 feet long, with an 8-inch face and 7 inches thick, have been found satisfactory. None of the ties received as first-class should have a greater face than 10 inches, and never less than 8 inches, and all first-class ties should be hewed from young, straight-grained, sound, live timber and be uniform in thickness, the faces straight and out of wind, bark chopped off, and the edges should not vary more than 3 inches from the straight; the ends of all ties should be sawed off square, and there should not be allowed a variation of more than 1 inch from the standard length of 8 feet, it being understood that a greater variation will render the ties second-class. The getting out of second-class cross-ties should be discouraged, and such ties should never be used in a main line; when received, second-class ties should be paid for at 50 per cent. of the price paid for first-class ties. Particular attention should be called to the requirement for length of ties, it being desirable that they should be uniform in length and the ends be square with the general line of the tie. Hewed ties should always be taken in preference to sawed ones, as the former will outlast the latter one-third and will hold track better. Cross-ties should be cut during the time when the sap is down, preferably from the middle of December to the middle of February, as the absence of sap will materially assist them in

resisting decay. In spacing ties in the track they should be laid not over 2 feet between centers, or 2 640 to the mile. When ties are delivered on the work they should be piled in square piles at designated points along the line, both ends exposed, and the party furnishing them should be required to handle them over again, if necessary, to insure a thorough inspection. Each pile of ties should be plainly marked with the full name and post-office address of the owner, thus preventing possible mistakes and trouble. Ordinarily, when track is being laid, it is done with a rush, and time is not taken to select the best ties for the joints; but this should always be done, as the joint, being the weakest part of the rail, should have the strongest support. Care should be taken to impress on the tie contractors the importance of uniform thickness for ties, and inspectors should pay especial attention to this point. If possible, all ties should be allowed to season under shelter for at least one year before being put in the track, and there should be permanent tie store-houses at convenient points along the line.

The chemical treating of timber for ties has, for a good many years, caused much thought and experiment, and it appears that creosoting, or forcing dead oil of tar into the wood, is about the only process that has thus far given much satisfaction. Chloride of zinc has been used in connection with creosote, and these chemicals, which are largely insoluble in water, penetrate a seasoned and open-grained tie in every part, and considering the increased life of the tie by their use, while the most costly, are not really the most expensive. The use of mineral salts for preserving timber has also been tried, but these being soluble in water are practically worthless for this purpose, unless the ties are in good broken stone ballast and the track well drained; it is said, however, that the use of mineral salts will harden the fiber of the wood, and thus offer greater resistance to the cutting into the tie by the rail. When ties are to be chemically treated better results are to be obtained by boring spike holes, and doing on them such adzing as may be necessary before they are shipped to the works for treatment; every part of the surface of the tie will then be equally fitted for offering effective resistance to the conditions producing decay, thus giving them the full benefit of the treatment. It pays better to treat soft wood than hard wood, and all wood to be successfully chemically treated should be open-grained and seasoned.

Tie-plates.—The use of the tie-plate has been largely restricted

to curves, as they are not of so much advantage on tangents, the tendency of the rail to cut into the tie on tangents being small as compared with certainty of similar action on curves. The Servis tie-plate (Fig. 4) seems to meet the requirements for a device of this kind better than anything else that has been brought to the attention of maintenance of way people, though there has recently appeared on the market the shoulder tie-plate which seems to possess some good points. The shape of the Servis tie-plate is such as to prevent bending, and the hatchet edges which enter the tie compress the wood on either side so as to render this part of the tie almost as impervious to the attacks of rot, induced by the action of moisture, as any other part of it. Besides protecting the face of the tie from rail wear, the plate binds the spikes together, each bracing against the other in such a manner as to prevent the working out or bending back of the spikes from the rail, which often occurs with spikes not thus laterally supported. In addition to the Servis plate shown in Fig. 4, the manufacturers make a plate which has in it a central flange; this three-flanged plate is wider, and therefore affords a better bearing for the rail and decreases the span between ties. The increased safety of trains, gained by the use of tie-plates, is a valuable advantage, and this,



THE SERVIS RAILROAD TIE-PLATE.

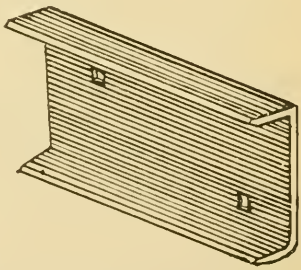
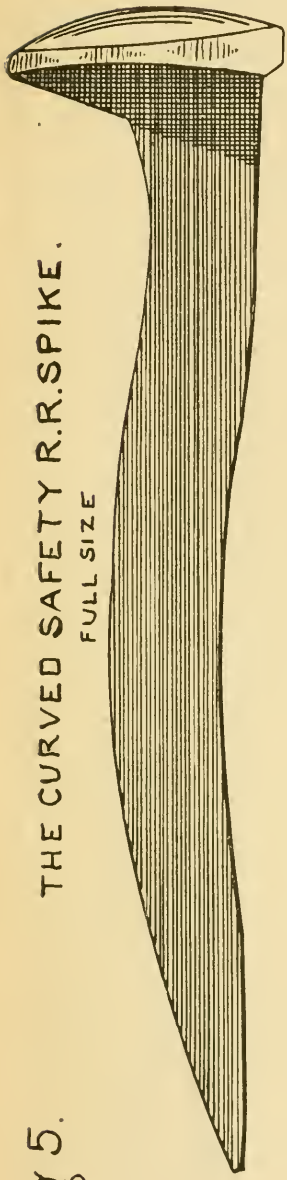


Fig. 4

of course, should be considered above everything else; another gain is in the rigid bearing afforded the rail and the effectual preventing of its canting. On curves, the plate causes the inside spikes to assist in sustaining the impact of the train, thus doubling the stiffness of the track. The manufacturers of the Servis tie-plate claim that by the use of this plate the life of the tie is doubled.

SPIKING.—The usual dimensions of track spikes are $5\frac{1}{2} \times \frac{9}{16}$ inches square, their weight about half a pound each, and they are put up in kegs of 200 pounds each. Their common defects are brittleness and imperfect points. In spiking track, the most important points to be attended to are the proper spacing of the ties and driving the spikes in such a manner that the ties shall be held in place, at right angles to the track and the rails in true gauge; to insure the latter, the track gauge should always be used when spiking the gauge side, the rail being held to proper position by a lining bar. The gauge should be kept about 6 or 8 inches ahead of the tie being spiked and should not be lifted until the spikes are driven home; gauges should be tested regularly and every morning when they are to be used all day, so as to insure a true gauge all the time. The two inner spikes should be set on one side of the tie and the two outer spikes on the other, as indicated in Fig. 5; this prevents the tie from slewing around and thus deranging the gauge of the track, as well as interfering with the proper spacing of the ties. The joints and centers should be spiked first, which will bring the rails to their proper position on the ties, which in turn will assist intermediate spiking. Each tie should be carefully gauged as spiked and as before indicated, the ties with the broadest faces being selected for the joints. In gauging ties, it is very convenient to have measured off on the handles of the mauls in the hands of the forward spikers, the distance from the outside of the rail to the end of the tie; this distance will then be gauged on the tie, when it will be lifted to the rail and securely spiked; the gauge is then used, and the loose rail held in place with the lining bar as previously indicated, loose gauge being given on curves, in accordance with directions of the engineer, the allowance for which is about an $\frac{1}{8}$ of an inch on a 2-degree curve, up to about $\frac{3}{4}$ of an inch on a 12-degree curve. This widening of the gauge should begin on the tangent, back of the P. C., the full amount of excess over true gauge being reached, by the time the P. C. is reached and continue all the way around the curve, running off from the P. T. in same manner as



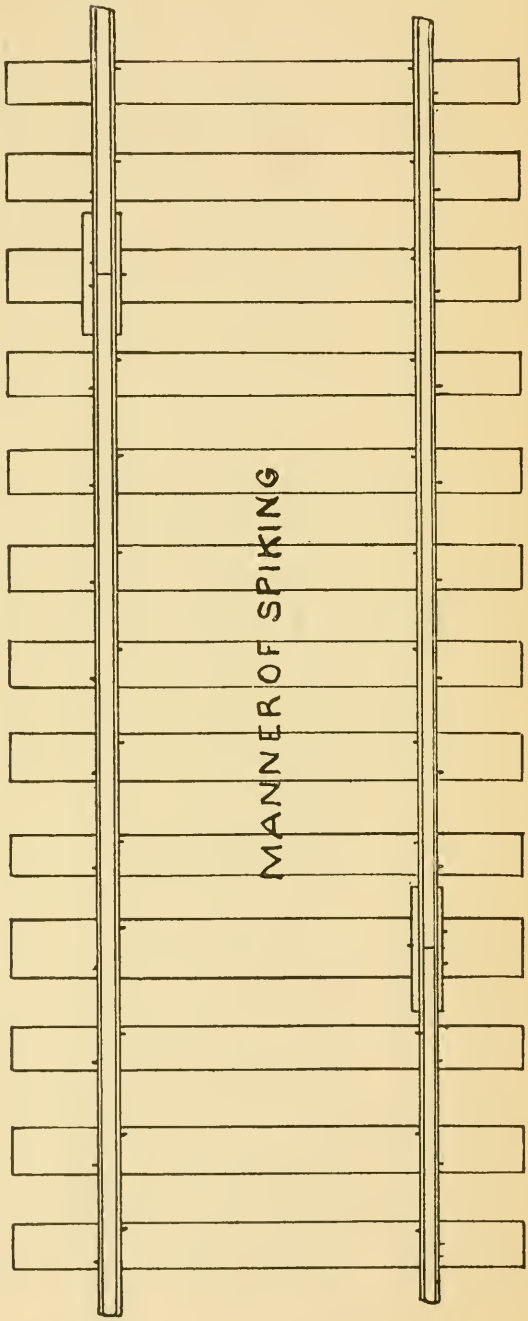


Fig 5.

back of the P. C. The spikes should always be driven home straight and at right angles with the face of the ties. When the foreman in charge of the track-laying work sees a spiker, when the spike is nearly home, strike the spike head laterally, which is done to make it lie snugly to the rail, he should at once check such imperfect work and put the man who does it at other work. The foreman in charge of gang of spikers should be experienced in this branch of the work, and by weeding out imperfect workers, can soon get together a first-rate gang of spikers. But no trouble will be experienced from carelessly driven spikes, if the tie has the spike holes bored into it, before laying; this is considered good practice, but rather expensive. For boring the holes quickly and accurately, a proper template should be made, by which the ties are marked for the borers, who should be provided with boring machines, by the use of which a hole, square with the face of the tie, is bored. The boring machines should be so arranged as not to cut the hole beyond the required depth, which should be slightly less than the length of the spike. The diameter of the holes should be about $\frac{1}{16}$ of an inch less than the thickness of the spike; this not only does away with the spike tearing its way through the timber and thus injuring its fiber to a great extent and causing it to be much more susceptible to rot, but it is said to increase the adhesion of the spike in hard wood ties, at least 50 per cent. But in order that the best results may be obtained, the spike should be flattened on either side of the sloping point, which will generally prevent its leaving the hole. The spikers should carefully avoid striking the rail with their mauls, as such carelessness often produces fracture, which sometimes causes the rail to break in two at such points, which is liable to produce derailment and serious accident. Spike mauls should weigh not less than nine nor more than ten pounds, and should be on straight handles, not less than 3 feet long. After considerable use, the face of the maul will become somewhat rounded, and when this takes place it should be sent to the shop to be redressed. The last blow on the spike should be only sufficiently hard to cause its throat to fit snugly on the rail; a harder blow will often fracture the spike in such a manner as to cause the head in a short time to break off, and leave the rail unsupported at that point. Foremen should not allow a spike to be pulled, especially in frosty weather, until it has been first struck a light blow to break the rust and loosen its hold in the wood. The filling of old spike holes with wooden plugs is bad practice,

for the reason that they will cause the spike in a short time to slip from its place; to fill the holes with sand is much better, and spikes driven in holes so filled will hold much more firmly. The best form of spike I have seen is the Curved Safety Railroad Spike (Fig. 5); this spike takes in the tie a position which enables it to resist the thrust of the rail against it, much more effectually than the ordinary spike can possibly do. I have seen in good condition, one of these curved spikes, which was said to have been driven eight times. The cost of the Curved Safety Spike is more than that of the ordinary spike, but it is better made, holds the track better, and, I believe, is worth more than the difference asked for it.

Rail Braces.—On sharp curves, where the rail has to resist the heavy lateral thrust produced by the impact of the trains, the spikes are not sufficiently strong to resist this strain unaided, and it is necessary to use rail braces. There are many styles of braces in the market, but I consider the Alkins Forged Steel Rail Brace (Fig. 6) the best, as this is so formed as to support strongly the head as well as the side of the rail, and is not too thick to prevent the spikes holding it from obtaining a secure hold in the tie. Next to this, I prefer a good snug-fitting cast brace, though the principal objection to this is that the thickness of the lower part prevents sufficient of the spike going into the tie, thereby failing to get the best service. Rail braces should be used on all curves over 6 degrees, and on this degree not less than three to the rail, and these should be increased as made necessary by increase of curvature and traffic.

Rails.—This is a subject on which much thought has been bestowed, especially recently, by the Rail Section Committee of the American Society of Civil Engineers, and I unhesitatingly declare in favor of their recommendations, until something better is proposed. Their recommendations are as follows:

First.—A 12-inch top radius as a standard for rail sections of all weights.

Second.—A broad head relatively to depth for sections of all weights, taking care not to go too far in either dimension, especially in very large or very small sections, or to endanger the flange cutting into the joint.

Third.—A ¼-inch corner radius as a standard for sections of all weights.

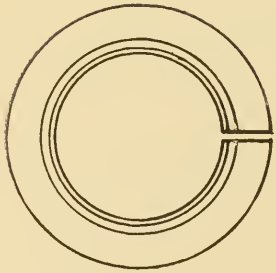
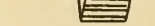
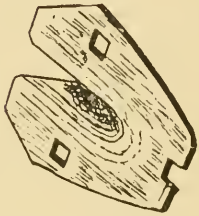
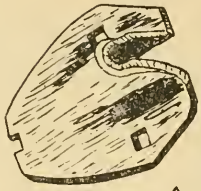


Fig 6.



National Nut-Lock.





Atkins Forged Steel Rail Brace.

Fourth.—A $\frac{1}{16}$-inch lower corner radius for the head, as a standard for sections of all weights.

Fifth.—That starting from a sufficient base width of head to give ample bearing for the joint or to conform to recommendation No. 1, the sides be carried up vertical, as a standard for sections of all weights.

It may be urged that our experience with rails formulated on a basis similar to the one indicated is limited, and to a large extent this is true; but the adoption of the proposed standard would be a boon to manufacturers as well as to railroads, for then rail-makers would be able to roll in slack time, and thus always have a stock on hand, and the railroads would be able to buy cheaper for this reason. Comparatively few railway managers have looked deeply into the subject of designing rail sections, and these, in addition to those who have not looked into it at all, would be glad to be able to buy rails of sections designed on the American Society standards. There are other managers who purchase rails only with reference to such weight per yard as may be required by their president and directors, and these would also be glad to select from such standards. Again, a set of standards formulated by the American Society of Civil Engineers need not be final; the Committee should continue the collection of data, and from time to time recommend such changes and modifications in their standards as might seem advantageous. The proposed standards need not in any way discourage experiment, for the reason that if any manager were to conclude that he had discovered a section superior to those of the American Society, he would try it if he could, which is just the same as he would do now. The weight of rail per yard for a road should be governed almost entirely by the amount of traffic of the line, but the weight of rails should keep well up with increase in weight of rolling stock. All rails should be carefully inspected as to line, surface, length, flaws, composition and weight. No excess of weight should be paid for, and rails should be selected and weighed by the inspector whenever he deems it necessary. The ends of rails should be sawed off square, and rough edges left by the saw should be filed down until the end conforms to the true section of the rail. Without this a neat fit of the joint cannot be had. The rails should be straight and not twisted, and the inspector should be authorized to reject a rail if it is struck or thrown violently. When the curves are 10 degrees and over, the rails should be carefully curved

before being placed in the track, as unless this is done slight angles will generally appear at the joints. Some approved rail-curving machine should be used, and rails should never be curved by being raised and dropped on ties laid for the purpose, as this manner of curving puts nearly all the curve in the center, and rails are also sometimes badly bent by this method. Nor is the rail-bender satisfactory, for the reason that the curving done by it is really a series of slight bends and not a regular curve as it should be. Rails with mitered ends have been tried to a small extent and have given good satisfaction, and the experiments thus far indicate that the miter adds to the strength of the joint. Rails should be handled carefully, and should never be dropped or thrown down. When unloaded from the cars, they should be slid down on skids sufficiently inclined to allow the rail to go gently to the ground, when it should be moved slightly to be out of the way of the one to follow, though this will not be necessary if rails are unloaded from both sides of the cars, which will be found a good plan. Where necessary to unload in one place, the rails should be neatly piled near the track convenient for reloading on cars. Rail is sometimes slid to the ground on skids and then pitched on the pile. This should never be done, as the rails are often injured thereby. If the ends of the rails have not been filed at the mill, they should be brought to the proper section with files, thus insuring a good fit of the angle bar. Fig. 7 represents standard 70 pounds rail of the Richmond and Danville Railroad, which is designed nearly on recommendations of the American Society Rail Committee. This 12-inch top radius gives a good bearing for the drivers, and the vertical sides offer less resistance to the wheel flanges.

Joints.—No absolutely satisfactory joint has thus far been found, but it would seem that one which would approach nearest to the principle of a bridge would be the best. The Fisher joint (Fig. 8) is on this principle, but has the objection of necessitating the removal of a sufficient quantity of the tie to make way for it, thus rendering the tie more liable to split and decay. If this cutting of the tie is not done it must be lowered, which is unsatisfactory. Longer spikes have also to be used with this joint, which is objectionable, this being necessary on account of the thickness of 1-inch channel beam through which spike has to pass. Ordinary spikes would not have sufficient hold. It is claimed that the strong vertical bolt holding the Fisher joint prevents creeping, on account of the rails butting up

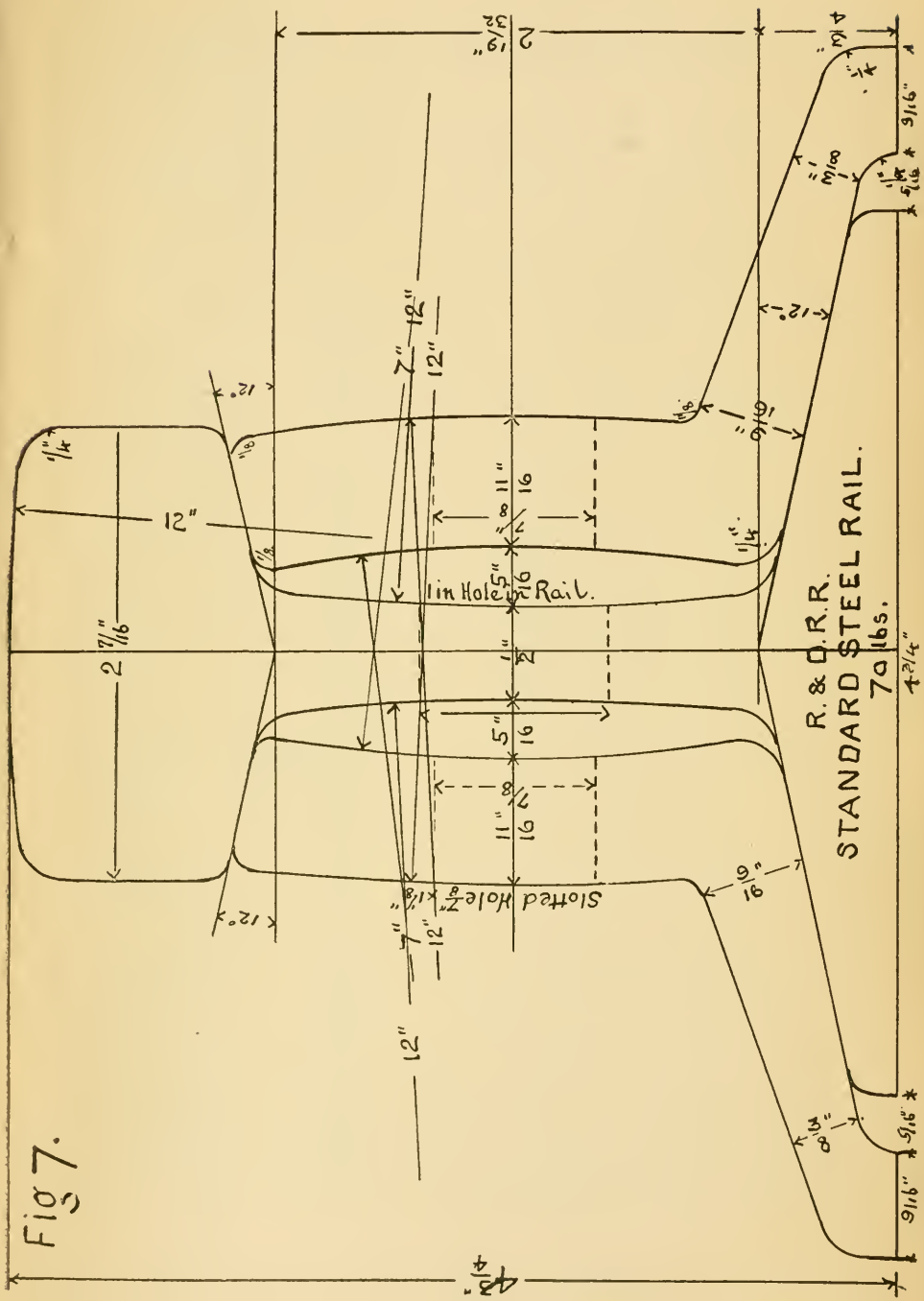

R. & D. R. R.
STANDARD STEEL RAIL.
70 lbs.
1 in Hole in Rail.
Slotted Hole 7/8" x 1 1/8"
12"
7"
2 7/16"
4 3/4"
9/16"
11/16"
5/16"
1/2"
7/8"
1/4"
3/8"
3/4"
2 19/32"

Fig 7.

against it, the corners being drilled off to receive it. This joint is still in an experimental stage, but is giving excellent satisfaction, and, in my opinion, is the strongest joint made. Its cost is the principal objection at present. There is a modification of the Fisher joint, manufactured by the same works, price of which is less, but the support is on a much smaller scale, and is used as an adjunct to the angle bar. It consists of a piece of steel, 2½ x 3½ inches, placed under the joint and held in place by a U-bolt, similar to the one in regular Fisher joint. The Fisher Joint Company has recently designed a small angle bar as an adjunct to their joint, which is, I think, an advantage, but of course renders the joint more expensive. This is said to be a powerful assistant to the angle plate, but first cost is greater than any plate 20 pounds per joint. The angle bar should be so designed as to give same strength as exists in the body of the rail. It is claimed for them that, while allowing each rail to contract and expand freely, the track as a whole is held against creeping. This class of joint does not give satisfaction, though a large majority of the lines continue to use them, I suppose, on account of cost of changing; but if for this cost we could obtain a

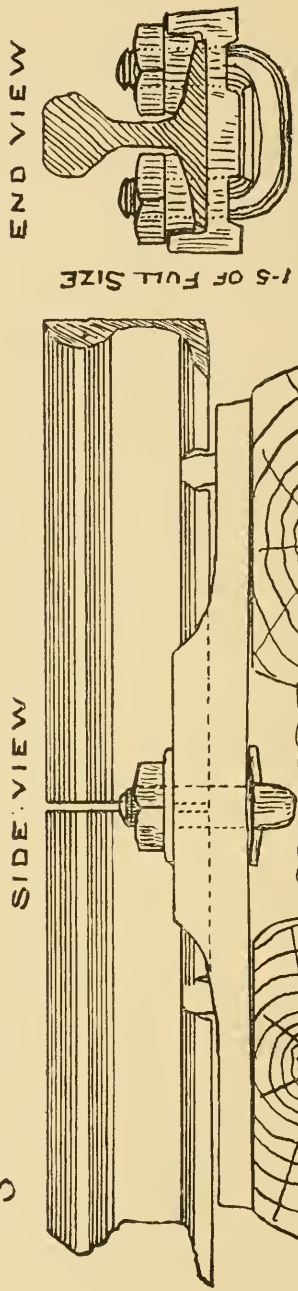


FIG 8.

THE FISHER BRIDGE JOINT.

better and more satisfactory joint, the saving in maintenance of low joints would in a short time cover the excess over the present unsatisfactory device. A good angle bar is shown in Fig. 8, though I would prefer the base of the bar to swing clear of the ties ⅛ inch. I am in favor of the supported joint, as being cheaper to keep up. The square-jointed rail gives more or less of an opening for the wheels to drop into, while the mitered rail (angles from 45 to 60 degrees) to a great extent obviates this trouble; still advocates of the joint claim that the supported joint being subjected continually to the sledge-hammer blows of the wheels, prevents to a large extent the springiness of the splice, thereby increasing their effect. But adding to the other advantages of the supported joint, the tie undoubtedly increases the bearing surface at the weakest point of the rail, which is a great advantage. With broken joints, track is easier and cheaper to keep up, and rides better, which are three good reasons why this mode of arranging the joints is better than to place them opposite. Track will also stay in line better with broken joints, especially on curves, the cause therefor being that as the weakest place in the rail is the joint, even joints bring the weak points of both rails together, while in broken joints the weak point is reinforced by the stiffness of the whole rail on the other side of the track. For the same reason it is undoubtedly wrong to claim that where track is allowed to run down or is put up on poor ballast even joints are best.

Bolts and Nut Locks.—Bolts are usually ¾ x 3¼ to 3¾ inches, as may be made necessary by distance through rail web and angle bars and thickness of locks to be used. Holes in rail for bolts are drilled usually circular, which provides for contraction and expansion in the rail; and in order that the bolt may not turn, an oval enlargement of section under head is made, and a corresponding hole is punched in the angle bars. Experience has proven that nut locks cause by their use a large decrease in expenditure for track maintenance, reducing cost for tightening of bolts at least one-half, and adding largely to the life of the bolt by holding it rigid. The use of spring nut locks has one disadvantage, which is that of producing the impression on the mind of the track-walker that the joint is tight, while in fact it may be loose and held tight by the spring; but some spring locks give fairly good results, though recently a good many of these have been found broken in the track. This, however, is probably due to improper temper of the steel.

The Harvey grip bolt, with one side of the thread square and the other inclined, used with the Stark nut lock, gives good satisfaction, though troublesome to handle, owing to a certain amount of care being necessary to prevent injury to the key. The National nut lock (Fig. 6) is about as good as any spring nut lock.

Track Laying.—The roadbed should first be brought to the true surface of sub-grade. When track laying is begun it is usually necessary for it to be finished quickly, and therefore as large a force should be put on the work as can be worked to advantage. For proper results the gang should be carefully organized and the work supervised by a thoroughly experienced man in this branch of construction, it being economy to pay well for such a man in preference to employing a cheap and therefore generally incompetent overseer. In each gang of a track-laying force, it is a good plan to pay secretly one good laborer an increase of from ten to fifteen cents per day, and by this means more work can be gotten out of the force than by all the driving and swearing a foreman can do. It is also a good plan to offer a prize to the men, which should consist of a small addition to the pay of each man, foreman as well as laborers, if a certain amount of work is done in a certain given time, but unless time is an object, of course this is not necessary, though on general principles it is a good plan any way. It has been generally observed that good work is done when the forces are in a good humor, and *vice versa*. The Negro labor of the South, when worked on a railroad, is the most satisfactory in the world. Heat rarely hurts the Negroes, and only give them plenty of water and keep them satisfied by fair, but at the same time decided treatment, and they will work well early and late, and the contractor or company employing them will get a good return for the money paid them. The men handling the ties should be carefully instructed as to the distance apart to lay them, and they should, of course, be in charge of a foreman who will see that the ties are regularly spaced as previously mentioned, the best ties being selected for the joints. When possible, the ties should be gotten out near the line of the road by contracts with the natives; this can be done in a good timber country, cheaply and satisfactorily, and the contractor should be required to deliver the ties near the roadbed and in piles of fifty every 100 feet. Where this delivery can be had the ties can be placed on the roadbed very cheaply. It should always be stipulated that ties so delivered shall never be placed more than 4 feet above or below grade, but where ties are necessarily placed on the side

of a cut, they should never be pitched over into the cut, but should be carried on to the roadbed and dropped in place.

Handling Rail.—It will be found that the handling of rail will be greatly facilitated by use of rail cars with rollers let in the platform or by strips strongly attached to the platform at each end of the car; these pieces should be 1 x 3 inches, and in length equal to the width of the car. An iron strap should be placed on each strip, on which the rails can be slid off the car. This arrangement will be found nearly as satisfactory as the rollers, will be cheaper, and will not interfere with the use of the car for other purposes. The track centers being in, it should be seen that the men handling the rail lay each one as near as possible to where it will lie when lined up, which will save labor in throwing the track and the possibility of kinking the rail, or tight joints when laid too far on one side of the center. Great care should be taken of the centers, and they should never be disturbed. The spacing between the ends of the rails should vary with the temperature. In warm weather they can be placed within a ¼ of an inch of each other; in cold weather they should be placed not less than ½ an inch apart. Unless the detail of rail spacing is carefully and intelligently looked after, especially in winter, trouble will come when the summer sun strikes the track, as if it causes the rails to buckle, serious derailments may occur. A very convenient way of spacing rails is done by holding a spike keg stave between the rail ends; the forward rail is then shoved up to the near one and the stave is broken off and used in succeeding spacing until used up. The joints and centers should first be spiked to gauge, which assists in intermediate spiking, and each tie should be carefully gauged before being spiked. In going around curves the joints of the inside rail will get ahead of those on the outside, thereby causing them to be greater and less than 15 feet apart for 30-feet rails; therefore, with every lot of rail, there should be a few 28 or 29-feet rails for compensation at these points. The heads of the bolts should always show on the inside of the rail. Square nuts are generally satisfactory, except where light rail is being laid, when hexagon nuts will usually be found necessary as occupying less space.

Alignment.—A railroad track when turned over to the Maintenance of Way Department, may be lined up true to the engineers' centers, but after these have been knocked up or covered up, the track gang will gradually lighten the curves at the ends and stiffen them towards the center; this causes the engine to deflect more gradually from the tan-

gents and adjust itself with less abruptness to the curve, and it would therefore seem that in laying out the line of a railway, transition or easement curves should be used. Curves of over 1 degree should be compounded from the beginning, increasing 1 degree for every 50 feet, until the P. C. of the full degree of curve is reached. The other end of the curve should be run out in the same way, this, of course, would increase the trouble and expense in protecting the location and laying it out on the ground, but it is probable that the cost of maintenance of track and rolling stock would be decreased sufficiently by this means to repay many times the expense of the first cost in engineering. Money spent in engineering is well spent, and the location of a line should never be hurried. A great mistake is made by the employment of incompetent engineers; the very best that can be gotten should be employed, and they should be well equipped and have plenty of time to do their work. By following this rule, railway managers would often save their companies subsequently much unnecessary expense, to say nothing of the numbers of roads that are to-day bankrupt and in the hands of receivers, on account of being poorly located. No matter how good a country a road is in, unless a paying load can be pulled over it, a receiver is a mere matter of time. No trackman's eye is sufficiently good to run a true curve without the aid of an instrument; therefore, permanent centers should be put in by the engineers. Iron plugs 24 inches long have proved satisfactory. The foreman should guard these most carefully, and they should be tested every spring after the frost is out of the ground and kept as near as possible in proper position.

Grades.—As a general law for choosing grades, Mr. A. M. Wellington, M. Am. Soc. C. E., in his "Railway Location," says: "Follow that route which affords the easiest possible grades for the longest possible distances, using to that end such amounts of distance, curvature and rise and fall as may be necessary, and then pass over the intervening distances on such grades as are then found necessary." This seems to me to cover the ground very fully, and if the engineer will adhere strictly to this rule, he will generally build a paying line for his company. A compensation of 0.06 feet per degree of curve may be properly made in grades.

Elevation of Curves.—The constant tendency of trains to go off at tangent makes elevation on curves necessary. A fixed rule is hard to find; general rules for similar conditions can be adopted, but these must

vary in special cases. Before any elevation is put in, the curve should be lined up true. The proper way to put elevation into a curve is to raise the outer rail, though a great many track men think it best to depress the inner rail, but the outer rail can be elevated more quickly and satisfactorily, and therefore more cheaply, and for these reasons raising the outer rail is more proper. Having decided on the elevation per degree for curves, the proper way to go to work is to line up the inside rail and pull it to a true surface; the outer rail can then be pulled by the track level to any desired height above the inner rail. In deciding on the elevation per degree for the outer rail of track, the speed of trains is the principal consideration, but slow, as well as fast, trains should be considered in this connection. The outer rail at the P. C. and P. T. should be the full elevation allowed for the curve, and this elevation should be run out on tangents at either end of the curve at the rate of 60 feet for each inch of elevation, if there is enough tangent distance. When short tangents occur between curves deflecting in the same direction, it is good practice to carry a certain amount of elevation throughout the tangent. Lighter curves require more elevation in proportion than heavy ones, and for a speed of 35 or 40 miles an hour not less than ¾ of an inch to the degree should be allowed, with a maximum elevation of 8 inches; it is possible that one inch to the degree will not be found too great, for 1, 2 and 3-degree curves. One very important point in connection with elevation of curves is, that whatever elevation is given should be uniform all around the curve. Foremen should be compelled to ride on the regular trains over their sections at stated intervals, to observe carefully the effect of certain elevation at certain points, and they should never stop working on a curve until it rides all right.

Ballast.—As track is heaved by frost in winter, that material for ballast which will absorb the least moisture is best. Broken granite in such sized pieces as will pass through a 2-inch ring is considered the best, though very satisfactory results are obtained from good, clean gravel. One of the objections to gravel, however, is that the pebbles composing it are nearly all round, and therefore it will not hold track in line as well as broken stone, the sharp corners of which stick into the ties and prevent lateral movement. Stone ballast is very hard on the bottoms of soft wood ties, rounding them in a very objectionable manner. In the North, slag and anthracite engine cinder are a good deal used, but it is said these are not very satisfactory, being dusty and not very

durable, though preventing vegetation and to a large extent heaving by frost. As great a depth of ballast as can be gotten, up to 2 feet, should be put under the ties, though 6 inches deep is of great advantage. Ballast should not usually be less than 8 inches deep, and with this depth the drainage of the track is found to be much improved. Another objection to gravel is that it can rarely be gotten clean, nearly always having with it a mixture of clay or loam, but if the gravel can be cleaned of these by washing out a good ballast can be gotten out at a less cost than broken stone, and nearly as satisfactory. Coarse sand is also fair ballast, and does not retain moisture to any considerable extent, but will be found not to hold track well, and labor on sand-ballasted track is considerable. The best cheap material at hand should be used in surfacing track on green banks, and expensive ballast not put in until the banks have settled as, if used, it will be lost. Surfacing or section gangs should never be allowed to use material from shoulder of embankment for ballast; this is an expensive as well as slovenly habit, and leaves the fill in condition to be much damaged by rains. Burnt clay has been used to some extent with reported good results. Clay and ordinary earth usually found in cuts is unfortunately much used in the South, for the reason that it is cheaper and more convenient than any other ballast. That which has the most mica or isinglass in it is the most objectionable, causing the track in wet weather to slip out of line. In summer, clay ballast packs very hard, and track work in such ballast is very difficult. In winter, frost causes clay-ballasted track to heave very badly; in spring, the track goes right down. Coarse grass or sod will temporarily assist very bad wet places, but about the best thing to be done then is to get a good lot of gravel under, which will generally relieve the trouble. Broken-stone ballast is very hard and troublesome to work, and less work in a given time can be done in it than in any other ballast.

Shimming.—As tamping in the winter is impossible, and when the ballast retains water, which it generally does from one cause or another, track is irregularly heaved and it becomes necessary to do something to bring it back as near as possible to a regular surface. This is generally done by putting blocks of wood of necessary thickness under the rail and on top of the tie. The thickness of these blocks should never be more than 2 inches, but in extreme cases, when more is necessary, a piece of 1-inch plank, the width of the top of the tie and not less than 2 feet long, should first be placed under the rail and the shim spiked on

that to the cross-tie. When the track has been raised by the action of the frost, the tie should be adzed only in extreme cases, it being much better, when possible and proper, to raise the rail on the opposite side of the track to the corresponding height. It is better for the foreman to split his shims out of a section of a straight-grained cross-tie, as shop-made shims of the required thickness are rarely at hand when needed. When track on curves has to be raised over 1 inch, probably the best way to do it is to drive a wedge-shaped block under the tie and thus raise the track to grade, but care should be taken to remove this blocking in the spring. During snow-storms a sufficient force should be kept out night and day to keep frogs and switches open for traffic, to patrol the track and see that everything is safe for passing of trains.

Track Maintenance.—In this branch of railroad work it seems often necessary for work to be done on Sunday; this is, however, not really necessary and can be avoided, except in extreme cases, by good management on the part of the roadmaster. That eminent trackman, the late Charles Latimer, M. Am. Soc. C. E., just before he died, wrote a paper entitled "A Day of Rest for the Weary Trackman," and advocated the men demanding Sunday as theirs; this would be unnecessary if the managers of railways would consider the matter, and it is much to be hoped that they will. The man who is always ready, day and night, to go out on the line should have a day of rest on Sunday, unless an extraordinary necessity causes him to work. In cases of a washout or a smash-up, Sunday work is a necessity, but generally it can and should be avoided. Parsimony in supplying men and materials for work on track is poor economy; allow a sufficiency of them, and when the track is in fine condition and well ballasted the decrease in track and rolling-stock repairs will be surprising. There are two things trackmen should never lose sight of: the one is that all work, and especially track work, should be done as far as possible in a regular and systematic manner, and the other is that no work should be done previous to that which is more necessary. Work carried on under these rules by a good, experienced foreman, and with a full force provided with a sufficiency of proper track tools, will give the best results. In the spring, when the frost gets out of the ground, the foreman should go to work to take the shims from under the rails, throwing the track into a good general line, taking out the low places and renewing rotten ties, preparatory to a

general surfacing and lining for the coming winter. Track should be put in the best of order once a year, and in the spring, summer and fall is the time to do it.

Renewing Ties.—This is a most important branch of maintenance, and should be done carefully and with good judgment. I have seen a foreman take ties out of a track, that would easily have been good for one or two years longer and leave next to them in the track, ties which should come out at once. It will, of course, be said that this man should have been discharged, and he was, but had not the roadmaster been watchful, this foreman would probably have injured the company with his ignorance and carelessness much more than he did. This foreman is not the only one who has taken ties out when they should stay in, and the roadmasters cannot watch these details too carefully. In renewing ties there is one rule which should always be carried out, which is, never disturb the bed of the old tie any more than can be helped, and a tie as near as possible the same size as the one to be renewed, should always be selected to replace the one to be taken out. When ties are renewed, the ballast is cut into from either side of the tie, and cut one side a little lower than the bed, the track raised a few inches, the spikes drawn, the tie pulled out and the other slipped into its place. From ten to twelve ties per day per man in renewing, is considered good work. In light ballast and under favorable conditions, better work can be done. A satisfactory way to test the life of a tie, is to strike it a sharp blow on the end, and if the wood seems soft, the tie should be renewed. Both ends of each doubtful-looking tie should be tested, and ties to be renewed should be marked with the blade of an axe, so there may be no mistake when the tie renewers come along. Ties supplied by natives should always be deposited at or near the top of a grade, so they can be easily run down hill to place on pole car; when distributed by train, they are usually thrown off near the place they will be used, thus saving the expense of handling a second time. Foremen sometimes allow their men to handle ties by striking a pick in them; this should never be done, as it often splits the ties and always leaves a hole by which water will enter and produce rot. At the end of each week's work old ties should be piled on the right of way, at convenient distances, for loading on cars, if they are to be hauled away, or well towards the outer edge of the right of way if they are to be gotten rid of by burning. It is very objectionable for the old ties to be allowed to clog up the ditches or remain scattered on the right of way. All ties should be renewed by the last of August.

Raising Track.—Some foremen raise one rail first to grade and then pull up the other on the same level with the track level; this is bad practice, as the raising of the second rail disturbs the material which is packed under the ties holding the first rail, thereby causing it to go down. Both rails should be raised together to a half or three-quarters of an inch above grade and tamped solidly. A good deal of track is still pulled to surface with raising bars, but these are fast being superseded by ratchet and friction track jacks, which are better and cheaper than a man swinging onto the end of a raising bar. I consider the friction jack the best, as it will hold the track at any desired height, while the ratchet jack has to be pulled to a notch before stopping. The new Stanford track jack seems to fill all the requirements for a friction jack, is very strong and durable, and holds track satisfactorily; this jack has been tried thoroughly on two divisions of the Richmond and Danville Railroad, and gives first rate satisfaction. When possible to have levels by which to raise track, they will be found a great advantage, the top of the grade pegs giving elevation to which top of rail is to be raised. On tangents the grade pegs should be set every 100 feet, and on curves every twenty-five, the elevation of pegs on curves being for lower or inside rail. While, of course, it is preferable to have elevations for raising track, a foreman with a good eye can put up track very well without them, but the levels save time and the track force should always be glad to get them.

Grassing.—One of the most troublesome things with which the trackman has to deal is grass and weeds, and if the grass is heavy and his force small, he will find that he can ill afford to take the time from his track to work on it; but as a neat as well as good track is desirable, the roadbed should be grubbed and the right of way mowed at least twice during the season, the last time about the first of August, after which time not very much more trouble will be experienced from grass and weeds, unless the soil is unusually productive. I have seen so much grass in the track on a poor road, that the engine could scarcely make any headway, owing to the slipping caused by the grass getting between the tread of the wheel and the rail. In grassing track outside the ends of the ties, the width of the cleaned space should be regular. Some days after the mowing and grubbing are finished the grass and weeds should be raked into a pile and burnt.

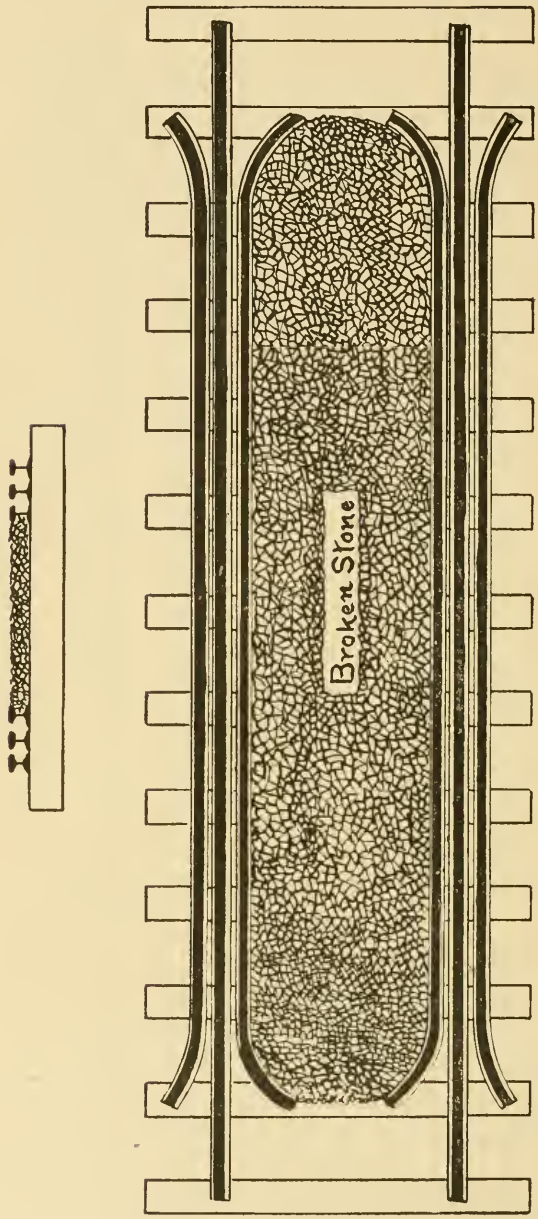
Tamping.—This is packing the ballast tightly around and under the

ties, and is largely done with shovel handles, which are generally preferred by trackmen to tamping bars or picks, though these latter are necessary for stone ballast and work to advantage in gravel, but the shovel handle works to good advantage in light ballasts. Ties should not be uniformly tamped from end to end, for the reason that owing to the weight of the rolling loads and flexibility of the tie, the ballast will pack more quickly at the ends and cause thereby what is called center bound track, which gives the train a very objectionable rocking motion. In order that this trouble may be avoided, ties should be well tamped under the ends up to within 1 foot on either side of the center, and these 2 feet in the center should be more lightly tamped than the ends. About the only satisfactory way to relieve center bound track is to raise the track clear of the old bed not less than 3 inches, and tamp solidly under the tie ends to within 1 foot on either side of the center, as previously indicated. A man should never attempt to tamp a tie alone, as he will thereby knock out on one side what he knocked under on the other. There should always be another man tamping the other side of the tie, their blows falling at the same time as nearly as possible, thus packing the ballast under the center by the concentrated action. The joint ties should be more firmly tamped than the balance, as being the points least able to resist the sledge-hammer blows of the wheels, should have most assistance to do it. In raising track the joint ties should first be tamped and put up a little high, then the centers, then the quarters, then the balance, and before leaving it, it will often be found necessary to finish off on the joint. Frogs should be carefully looked after and kept well tamped, especially the points, as this is also a point which is unable to resist effectually the constant hammering of the wheels.

General Work.—By the time the foreman has renewed all of his rotten ties, got his track in good line and surfaced and ready for the winter, it is getting well into fall. He should after this disturb the track no more than is absolutely necessary. The ditches should be cleaned out and the berme ditches and culverts cleared of leaves, trash and such other obstructions as would likely choke the openings in a heavy rainfall or melting of snow; the frogs and switches should be kept clear of ice and snow, doing which is greatly aided by liberal sprinkling of salt on the parts most liable to be affected. When the track heaves badly, it should be marked with paint, and in the spring such places should be dug out and filled in with clean sharp sand or gravel. Such parts had

better not be disturbed in winter by other work than putting in shims to raise low places. All bolts should be tightened at least twice a year, and if this is done the track-walkers can take care of them after that. When steel is furnished for renewals the rotten ties should be renewed and track resurfaced before laying it, and all steel should be laid together in spots in preference to using it for general renewals. In renewing rail, if lengths are put in different from those taken out, all of the bolt holes should be bored and joint made up as good as any of the others. In Northern climates, snow causes a good deal of trouble to the track and train men, and special appliances are used for removing it; there are also many miles of snow sheds in the Northwest for protecting the track from the heavy drifts. The section foremen should carefully watch all points where slides, washouts or other troubles are likely to occur, and they should at once remove any rocks, stumps or masses of earth which are liable to fall or slide on the track or are too close to it. They should also take every precaution during heavy storms to prevent accident, and all hands should then watch the road, and especially at exposed points. The track on banks next to bridges and trestles should be kept well surfaced, not allowed to get lower or to be raised higher than the regular grade of the structure, it being very injurious to the rolling stock to receive the shock caused by such inequalities in elevation. When trestles go down, wedges to the height of 4 or 5 inches may be used to raise them, but beyond that height the wedges should be removed and substantial blocking put in their place. Shimming of trestles should always be done under the sill, and no foreman should block up a trestle by putting shims under the stringers or ties.

Road Crossings and Fences.—Where road crossings at grade are unavoidable, a very satisfactory and durable crossing is made by two rails, spiked down on each side of the main rails, allowing between bases of rails sufficient width only for the spikes ; then fill in the road-bed on the crossing and approaches to the crossing with good stone ballast, and in a short time the passing of teams will pack the stone and make the crossing a first-rate one. (Fig. 9.) A very fair crossing is made by using 2-inch oak plank in place of the ballast, the plank, however, extending not further on either side of the track than the ends of the cross-ties. When possible, small brick culverts should be used for the ditch under the road leading up to the crossing, though terra cotta pipe properly protected at the ends, makes a very cheap and good culvert.

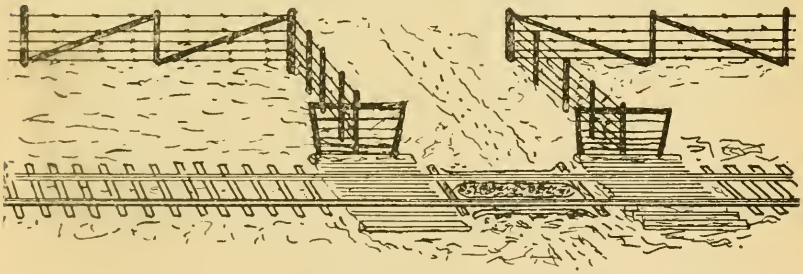


HIGHWAY CROSSING.

Fig. 9.

When the ditches have to be planked over, they should be well looked after and kept in good repair. A hole in the planking, allowing a horse's foot to get through, resulting in a broken leg, will cost the company annoyance and money, as it is always the finest animals that are killed and injured by a railroad. The slopes of the approaches to the crossing should be gradual, and not ordinarily over 1 foot in 10, as in steeper grades heavily loaded teams may get stalled, possibly resulting in an accident. Farm road crossings need not be over 10 feet in width, but public road crossings should always be sufficiently wide for two teams to pass. Width of crossings in towns and cities is usually the same as the width of the streets. In cities, and in some towns, especially where a good deal of shifting of cars has to be done over the crossings, the most desirable crossings are above or below grade, which not only saves risk of accident and delay and dissatisfaction to travelers, but the expense of gate-men, an item of considerable expense, is dispensed with.

Fences.—For a cheap and durable fence, Fig. 10, one of wire, nailed to locust or cedar posts, is unsurpassed. First, its cost is less than any other fence; it can be quickly put up and kept in repair at small expense. A heavy barbed wire is preferable to any other, and a plank 1 x 6 inches, oak or chestnut, should take the place of the top wire and be nailed on the top of the posts. This plank will act as a brace, and considerably strengthen the fence. The height of the fence should be not



BARBED WIRE RAILROAD FENCE.
Fig. 10.

less than 5 feet, and the two lower wires should be not over 6 inches apart, which will prevent small animals from creeping through. Posts should be spaced 12 feet apart, and every twentieth post braced

both ways. Posts should not be less than 8 feet long nor less than 4½ inches at the small end. Five wires should be used, but when the plank at top is used only four are necessary.

Force and Length of Section.—For convenience in maintenance, a railroad track is divided into sections, each of which is taken care of by a foreman and gang of laborers, one of whom should be rather better and paid a little more than the balance, and this man should be capable of handling the force intelligently during the foreman's absence. A large majority of foremen think they should do nothing more than stand by and see their men work. This is wrong. The foreman should be the foremost man in a great deal of the work done on a section, and he should insist on his men keeping up with him. In this way twice the amount of work can be gotten out of the force. In addition to the section forces, there are usually in summer several large floating gangs on each division, as well as two or more material or general work trains, and their assistance is usually necessary for getting the track in good shape during the season. The extra gangs usually help out in getting up the worst track, and the material trains haul all the material and clean out ditches, which latter work the section forces usually have not time to do. The extra gangs should also do a good deal of ditching. The size of the force and length of the section depend on a good many points, amongst which are, amount of traffic, number of depot yards and kind of ballast. Usually, on first-class single-track lines, the sections are 6 or 7 miles long, and force allowed is about one man to the mile. The foreman and his men should all live in section houses, located near together, so in case of a sudden call the men will not be scattered. The headquarters of the force should always be near the center of the section, and by this means the force is never more than 3 or 3½ miles from any point on their work. Where very large yards occur, a special gang will be constantly required for its maintenance, otherwise it may be neglected for work considered more important on the main line. The size of the section force may be decreased in winter, in a section where snow is not troublesome, and looking after switches, patrolling track and picking up low places is the principal work, but in summer a foreman should have from 10 to 12 men in his force; and he can, if he understands his business, work them to advantage. In both winter and summer, at least two track-walkers are necessary, and they should be in excess of the number of men allowed

as laborers. Track-walkers should be provided with a light spiking hammer, a flag and track wrench, and should be expected to drive down loose spikes and tighten bolts. The usual idea is that any sort of laborer will do for a track-walker, but this is a mistake; he should be a man of experience and judgment, and superior to the ordinary laborer, as his position is of a good deal of importance. In good weather, when there is no danger of accident, the track-walker may join the regular force during the afternoon of each day, but in stormy weather he should keep on the move, if necessary, night and day, and watch carefully all places where accident is liable to occur. The night track-walker should be provided with a lantern and torpedoes, and should arrange to get over the sections just ahead of the trains. In uncertain weather no job should be started that cannot be finished in the one day, and no work should be hurried over and left half done. A foreman who persistently does this should be discharged. The necessity for judgment in a foreman in handling his force is fully as important as a good knowledge of the work to be done. He should know how to distribute his men so as to get the best results from the work of each; it being best, if possible, for each man to have a small job which he individually must complete. The foreman in handling his men should be firm and just. The habit of swearing at the men, so generally indulged in by foremen, is useless, and the best discipline cannot be maintained by profanity. No man should ever be discharged without sufficient cause, and when any doubt exists as to the justness of the action, the laborer's as well as the foreman's side should be heard. I am no advocate for taking back discharged men, but where men have committed a fault for which they should be discharged, if their previous records are good, and they try to fulfill the duties properly, the foreman can sometimes place their shortcomings before them in such a light that they will see where they have been wrong, and if given another chance, will often make first-rate men; but where men are negligent and willfully disobedient, or get drunk periodically, no matter if off duty, they should be discharged and not allowed on the work again in any capacity. The foreman at all times should have impressed on him the responsibility of his position, and he in turn should endeavor to impress on his men their individual responsibility. All should be encouraged to take a pride in the work, and they should understand that good work will be substantially appreciated. A short, brusque manner, or orders roughly

given, on the part of a superior, while it may be entirely unintentional and caused by absorption in duties, will very often cause subordinates to feel that they are badly treated and thought to be beneath the notice of the superior, while a kind word and care for the welfare of the men will not only make subordinates have greater respect for those above them, but they will work better and be better men themselves. Foremen should be taught to think, and if they don't understand, to ask the reason why. All track-men should watch and study the work and endeavor to understand all about it. In the whole track force, from the roadmaster down, each man is indirectly dependent on the other, and all should work for general advantage, and endeavor to improve the condition of the work in every way; thus, if not at once, in the long run improving themselves.

Switches and Frogs.—Where it is necessary to pass from one track to another, switches and frogs are used. Switches may be divided into three classes: stub, split and Wharton. The first is the simplest and is still a good deal used in yards and by poorer lines, but the two last-named switches are rapidly replacing them. Split or Lorenz switches are undoubtedly the best for yards and the Wharton the best in every way for the main line. In locating a siding it is better to place it on the outside than the inside of a curve, as it not only looks better but does not obstruct a clear view of the main line by engineers of approaching trains. Switches should not be placed near bridges nor on high banks, as in case of accident, on account of them, the near proximity to the abrupt changes of elevation will cause the trouble to be more serious. Where sidings are on grades, beams of wood are sometimes placed on the lower ends to prevent cars being blown on to the main line. A much better precaution is to put in a ground lever thrown on the outside rail at the point of curve back into the main line, and cars blown towards it will by it be diverted from instead of towards the main line. Where a great number of trains are handled, sidings located as indicated in Fig. 11 will be found of great advantage and far superior to the ordinarily located sidings. Suppose passenger train has to pass freight trains going either way; freight trains going from *A* towards *D* enter siding at *A*, and *vice versa* those going from *D* towards *A*, enter siding at *D;* passenger train passes and instead of one of the freight trains having to wait for the other to get out of the way as with the ordinary siding, they pull out on the main line

simultaneously, the one on *AC* going towards *D* entering main line at *C* and the one on *DB* going towards *A* entering main line at *B*. The signal station should be located at some convenient point between *B* and *C*, and trains not being allowed to enter the main line at *A* or *D*, the operator has everything immediately under his eye and no movement can take place unless he is cognizant of it. The Wharton switch for safety and durability is unsurpassed, and with a spring frog it presents a practically unbroken main line and is really not a switch until thrown for use. The only drawback to the Wharton switch is its cost, which is several times that of the split switch, but as it lasts several times as long, this is not considered an objection; it is subjected to no wear except when in actual use and repairs on it are little or nothing. When accidents occur with the Wharton, an investigation will often show that the guard rail at the point has been placed too far from the main rail, thereby not being effective in forcing the wheel to mount the elevated rail, and this rail should always fit very snugly to the main rail. Where a Wharton switch has been properly put in and is in good order, no accidents need be feared from them, unless a train passes over them too rapidly. Lorenz or split switches, as previously noted, are very good for use in yards, but the guard rails must run close up to the points and the ends next to the points must be flared by an angle and not by a moderate curve. The split switch is unsafe for main-line use, unless trailed. When faced, there is chance for a sharp wheel flange to get between the rail and the main rail and cause a bad accident. Split switches, however, are largely used in main lines, especially in the North, but the number of accidents reported from them far exceeds those by the Wharton.

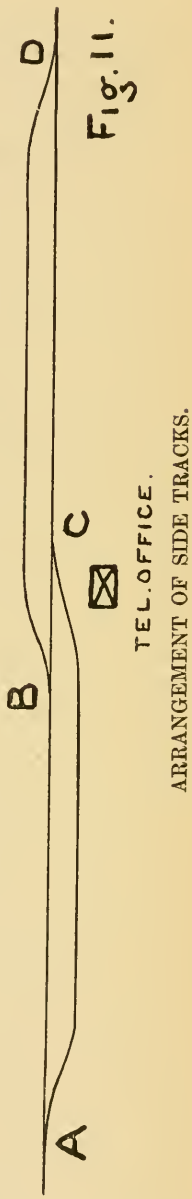


Fig. 11.

ARRANGEMENT OF SIDE TRACKS.

Frogs.—Stiff keyed frogs for general side-track and yard use, give good satisfaction, as they wear well and are not liable to get out of order if properly looked after. In the main line, a spring rail frog, which represents an unbroken bearing, is the best, as it saves wear and tear of the rolling stock and the frog itself.

Switch Stands.—These are necessary for throwing the rails of the switches, so trains may pass from one track to another. There are many kinds of switch stands, but the best are the High Switch Signal for use on main lines, the Ramapo Automatic Switch Stand for general use around stations and in yards, and in crowded yards the low Ramapo Stand.

Inspection.—The time of the engineer of maintenance of way is largely taken up with new work, but as often as possible he should ride over his track and examine it as carefully as possible, making notes of irregularities and noting if imperfections previously noted have received attention, and he should as far as possible keep informed in regard to the work done by each force. He should also walk over the track with the foremen as much as possible, talking to them and discussing their work with them, thereby encouraging them to greater exertions and improvement. Supervisors should always make weekly inspection of their work and section foremen daily inspection. All inspections should be made regularly, and when any defects or obstructions of the track are discovered by the track-walkers, they should report same at once to their section foreman, and if the trouble cannot be promptly and properly attended to with the section force, the foreman should call on the supervisor for such additional help as he may deem necessary. Inspections should not be merely glances at the track from the rear end of a rapidly moving train. Everything should be measured that cannot otherwise be fully known. The track foreman should never know when to expect his superior. Systematic inspection is generally necessary to discover bad practice and to properly instruct the foremen and see that they carry out orders. The supervisors should also regularly inspect the section tools and tool houses and see that each gang has its full complement, and that all are kept carefully and in good shape, and that none are lost or wasted. All tools and supplies should be issued to the track foremen on their requisition, as it is better that they should never complain that they cannot do certain work for want of tools necessary, but supervisors should see that all such tools are actually in possession of

the force or have been used as indicated in their reports. Careless and wasteful handling of tools and supplies costs the company a good deal of money.

PREMIUMS.—A good many lines offer premiums to their supervisors and foremen and men, for good work, careful and economical handling of their tools and materials, etc., and no money is ever better spent by the company, nor is there any way in which they get a better return than by such expenditures. The men are constantly encouraged, a friendly rivalry springs up between them, and by each trying his best all the time a high standard of excellence is reached. The premium may consist of one month's pay, and should be offered to the men as well as the supervisors and foremen, and this will make the men not only do good work, but endeavor to make full time, increasing the amount to be paid them for the premium. One inspection should be made per annum on a slow-moving train, and the judges should consist of the chief engineer of the line and certain of the division superintendents and engineers of maintenance of way, and always the latter officers of the division on which the inspection is being made. This annual inspection is of great assistance and encouragement to the men, and shows them that actual merit and good work win, and not favoritism in any way. In addition to awarding prizes, where men have done especially well, though not as well as the prize winners, they should be favorably mentioned. A very good plan is also to offer a prize of $10 to the wife of the section foreman who keeps the neatest section house and grounds, and, in the event of his being unmarried, to the foreman himself; in addition to the prize, the foreman should also be given a certificate, setting forth a degree of excellence attained by him, which entitled him to the prize. These annual inspections will very soon necessitate grading the foremen into such classes as they belong, and the first-class foreman should be better paid than the second-class, and so on. In addition to general excellence a prize should be offered for the greatest improvement in any one section on a division. It may be argued that the men should give the best work for their regular pay, but on trial it will be found that the premium system has many decided advantages, and will cause the track to be gotten up and kept in better shape than without it, and a road has only to try it awhile to adopt it for a regular thing.

In conclusion, I will say that only the best of all classes of workers,

from the chief engineer to the laborer, should be employed. All should be encouraged to do their best, and should be made to feel that careful and first-class work is appreciated, and such work should be valued highly and well paid for. Incompetent and careless men should be gradually weeded out, and a high standard of excellence in forces and work aimed at continually. The manager who can keep his men interested in their work and the property of the road will find that his line will be successful, not only in paying well, but in satisfying the inhabitants of the country through which it runs, which means the gradual working up of a good local business, which is beyond the reach of competing lines. All of these satisfying conditions can hardly be expected on one line, but the road coming nearest to them will stand the best chance for making money, which is what we all want to do.

DISCUSSION.

George R. Hardy, M. Am. Soc. C. E.—I would hardly feel like discussing a paper of which I only heard a part, but one point attracted my attention, and is perhaps worthy of notice; that is, the question of obtaining the alignment of a location on a curve. I have had considerable experience with tracks which have been originally located with an effort to ease the track curves at the ends, by doing it in the alignment, which always produces more difficulty than is gained. The alignment should be so given that it can be restored, as hardly any roads in this country are prepared to stake out all their curves and establish permanent points. It will be found necessary, in order to replace the alignment, to intersect the tangents, and when this is done, if it is a simple curve, it can be easily obtained by measuring the external distance of that curve. If it is a compound curve there is more difficulty, and yet the solution can be obtained there; but if it comes to a curve in which the compounding is of a different degree at each end, it is impracticable to restore that curve without fixed data. Now, the method of easing the curve is one of the simplest things. I do not say one of the simplest in the sense of requiring no mathematics, but it is a matter which can be very easily solved by those who understand it. The practice of to-day has been very well developed by the methods used on the Pennsylvania Road, and the same practice has been still further developed and simplified on the New York and New Haven Road. The rules which are adopted there are fundamental, and the results are the very best. They produce the parabolic curve of the third degree for adjusting the ends of the curve. Whenever the track is to be

relined the center curve can be easily found, and the adjustment can be staked out again.

In my practice I find it better to depress the inner rail just as much as the outer rail is raised, preserving the center of gravity of the train in better relation to the line in passing around the curve. As regards the amount of elevation, it seems to me that the figure mentioned, 8 inches on any curve, is excessive. The highest that I allowed on heavy traffic was 4 inches on as sharp as a 6-degree curve. If that was not sufficient for the speed, it was necessary that they bring the speed down. The fact of the matter is, in regard to the elevation of the curves, if we follow Professor Henck's rule we will get an elevation in which if we take the speed in miles per hour as the length of a chord, we shall find that the middle ordinate equal to the elevation would be that of a chord about $1\frac{6}{10}$ times the speed length. I find that satisfactory practice has been to take the speed in miles per hour and add 50 per cent., which makes 60, at a speed of 40, and take the middle ordinate of that distance for the elevation. Track-men can easily apply that rule and determine the proper elevation, and the tangent points of unadjusted curves at the ordinate would be about $\frac{3}{4}$ths of that on the curve, and would be run out in a distance 50 or 75 feet, according to the height at the tangent point.

Robert L. Harris, M. Am. Soc. C. E.—I consider that Mr. Hall, at the beginning of his paper, is too explicit upon the slopes to be given to earth cuts, *i.e.*, 1 to 1. During long and varied experience in many parts of North America I have found the general slope has been $1\frac{1}{2}$ to 1. The slope that should be adopted at any locality depends on the special character of its material. I have seen clay that stands nearly vertical better than at any inclination.

A surface ditch placed at three feet from the slope line is mentioned; usually this is too close; the top of the slope will split off and the formation will be destroyed, especially in clay countries, or where there is frost or great heat.

Pole drains, as used in some parts of Canada, constitute an efficient, as well as economical, means of under draining new wet cuts in "a wooden country," where tile drains or broken rock are not convenient or are too expensive. Deep ditches are dug, in which bundles of saplings are placed continuously and covered; this mode works well also in draining wet slopes.

Julien A. Hall, M. Am. Soc. C. E.—I have noted what Mr. Hardy has to say in regard to curve location, and that he thinks there are as yet hardly any roads in this country that are prepared to stake out their curves and establish permanent points; the Richmond and Danville is doing this on all newly constructed lines, and it is the intention of this company to re-locate points on all of its lines as rapidly as possible.

I also notice that Mr. Hardy thinks it proper, in elevating curves, to

depress the inner rail as much as the outer one is elevated; I think he would hardly succeed in getting any practical track-man to agree with him in this. For the reasons outlined in my paper, I think that all of the elevation should be put in the outer rail of the curve, and I also think that the inner rail should hold to the exact grade of the track, no matter what that grade is, and never be put below it.

I note that Mr. Harris thinks that I am too explicit in my specification of cut slopes; in addition to reasons shown, where cuts are taken out in good stiff material, it is sometimes many years before a 1 : 1 slope will wash to the natural slope of 1½ : 1, and I know cuts now, on the Richmond and Danville that have washed considerably steeper than 1 : 1, and although, as stated, I believe it is less expensive to take out cuts, in construction to 1 : 1 slope, I also believe it is less expensive to clean out a ditch at the foot of a 1 : 1 slope than to take out the cut 1½ : 1 when the construction is in progress.

Now, as to surface ditches, while I named 3 feet as being the proper distance for these from the top of the slope, I am frank to admit that in some material more space is necessary, yet in all material that stands well, I think the distance of 3 feet is sufficient.

I think well of Mr. Harris' method of draining wet cuts with bundles of saplings; this is a modification of the farmers' drains of which I spoke, and would, I think, be very effective. A simple wooden box, with a 4-inch square opening, sunk in either ditch of a wet cut, is also an excellent plan and will not fail in its purpose if properly put in; this has been tried very recently on one of our new lines, in a cut where the men and mules could scarcely get about, and its success was remarkable.

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